MARINE WORKS

ERNEST LATHAM

MARINE WORKS

Example of Erosion at Foot of Masonry Sea-Wall, 1921.

[*Frontispiece*

16/- Net

MARINE WORKS

A PRACTICAL TREATISE FOR MARITIME ENGINEERS, LANDOWNERS AND PUBLIC AUTHORITIES

BY

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With a Preface by

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AUTHOR'S NOTE

THE present volume is the outcome of some sixteen years' experience, and it is hoped that it may prove of real assistance to those engaged in the many and hazardous enterprises comprised within the term "marine works."

I desire to acknowledge the facilities given me by my partner, Mr Alfred Edward Carey, M.Inst.C.E., in respect of access to certain personal records, and the valuable assistance afforded by Mr R. Yates in preparing the subject matter for the press. I have also to thank the proprietors of several Technical Journals for granting me permission to reproduce certain articles and illustrations which have appeared in their publications. Each specific acknowledgment will be found in the form of a footnote to those sections of the book where such articles have been drawn upon.

E. L.

WESTMINSTER,
August 1922.

PREFACE

By C. LE MAISTRE, C.B.E., A.M.Inst.C.E., M.I.E.E.,
Secretary of the British Engineering Standards Association.

THE fact of my having known the author during the greater part of his career enhances the pleasure I feel in accepting the task of writing a short preface to this volume and recommending it to the engineering public generally.

The methodical manner in which he has dealt with his subject throughout the book bears evident traces of his earlier training and experience in the work of standardisation. He has dealt most successfully with the question of sea and coastal problems, which may be said to come within the category of maritime engineering, and constitute for a consulting engineer a somewhat precarious field of enterprise where the measure of success is more often gauged by the fewest number of failures rather than by the greatest number of successes.

The volume commences with a study of wave action, passes on to the difficulties of salvage operations and to questions of construction and maintenance of marine works. One of the very interesting subjects the author deals with is that of tidal hydro-electric undertakings as distinct from the purely river or lake schemes which occur so frequently on the continents of America and Europe. The problem of obtaining continuous power from the latent energy of the tides has baffled successive generations, and forms a peculiarly interesting border-line between the functions of the civil and electrical engineer. So far, apparently, no really practical scheme has come into operation, the cost of such installations having hitherto proved their greatest enemy. Schemes are constantly being heralded in the press; amongst them the Severn, the Medway, and the Welsh Cleddaus schemes may

be mentioned. The great difficulty, however, in all these proposals is the maintenance of continuous power, and in this connection the low-level reservoir system referred to by the author would seem to offer a distinct solution where the topographical characteristics are favourable.

It is with regret that one sees some of one's illusions dispelled, though perhaps rightly so, as one is brought to realise more fully that our British rivers, speaking generally, cannot be utilised at one and the same time to provide both power as well as navigation, for despite all the ingenuity which may be displayed in the construction of barrages, locks, and lifts, it is difficult to have it both ways.

A critical analysis of the advantages and disadvantages of deep-water quays is given by the author. This is a big controversy, and the chapter dealing with this subject may well be recommended to those in commercial circles who can appreciate a general and broad review of economical facts presented in not too technical a manner.

Finally, I cannot help but feel that this work, written in such a clear and original manner, will prove of very considerable interest to a large circle of the engineering public, to whom I confidently recommend its perusal.

C. LE MAISTRE.

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MARINE WORKS

CHAPTER I

WAVES: THEIR FORMATION, ACTION, AND EFFECT[1]

Relations between Height and Fetch — Considerations of Range — Velocities and Periods — Classification of Waves — Pressure Exerted and Damage Caused by Waves.

THERE are not many references to wave action in existing text-books, but Professor Ernest Matthews, in his book on "Coast Erosion and Protection," deals in his first chapter with the subject. Probably no other volume will be found to contain such a valuable collection of photographs, taken during storms, under obviously difficult conditions, and the reader interested in this subject is advised to add this work to his collection.[2]

The theory of wave action is somewhat indeterminate, and there is a valuable contribution to the subject given by Mr C. Colson in an appendix to his book. He refers, in the first instance, to a formula of the late Mr T. Stevenson, used

[1] Reprinted from an article in *Water and Water Engineering*, 20th January 1922.

[2] The subjects of wave formation and action are also dealt with in the following books: "Harbours and Docks," vol. i. (L. F. Vernon-Harcourt), Clarendon Press. "The Sea Coast" (W. H. Wheeler), Longmans, Green, & Co. "Manual of Hydrology" (Nathaniel Beardmore), Waterlow & Sons. "Notes on Docks and Dock Construction" (C. Colson), Longmans, Green, & Co. "Tidal Lands" (Carey and Oliver), Blackie & Sons. "Maintenance of Foreshores," Crosby Lockwood. "Wave Impact on Engineering Structures" (Prof. A. H. Gibson), Institution of Civil Engineers.

for the purpose of determining the height of waves. The formula quoted is

$$h = 1.5\sqrt{d} \qquad (1)$$

where h is the height of waves in feet, and d is the length of exposure in miles, or the "fetch."

The height of waves is deceptive, but in conjunction with other physical properties has a direct bearing on the dynamical force which the wave is capable of exerting. If this formula is applied to a "fetch" of 2,000 miles, a wave height of some 60 ft. is obtained. In the author's opinion, even under the worst conditions, there is no reliable record of any wave of such height. It is not clear from Stevenson's formula whether the height referred to is the vertical interval between crest and trough or the height of wave above mean sea level, but it is presumably the former. Mr Colson had apparently doubt about this formula, as he modified it for short fetches. This indicates appreciation on his part of the most important but obscure function in wave action, namely, the phenomenon known as "range."

Range may really be regarded as the potential damaging effect of a travelling wave, and is therefore a function of its velocity, height, and length. While, owing to the nature of the sea bed, or to the existence of lateral constrictions or to other causes, the ratios between these variables may alter, the range of the wave remains virtually the same until its potential energy commences to be dissipated.

The author suggested in 1919 that probably an attempt might be made to express the term "range" mathematically. As a result, the following suggested formula was put forward by Mr A. E. Carey (see p. 227, *Proceedings Inst. C.E.*, 1921, vol. ccix.):—

$$R = \frac{\sqrt{V} + fH}{L} - \frac{\sqrt{v} + fh}{l} \qquad (2)$$

where v = velocity in feet per second,
h = height in feet,
l = interval of waves in feet,
and f = a factor to be determined by experiment.

V, H, and L would represent the same components of

the wave after passing a harbour entrance or other constriction.

R would then be the range expressed in units to be defined by experiment.

The higher the value of the range factor in a wave the greater the potential energy of the wave as regards its capacity for doing damage.

From the above considerations it is obvious that actual observation on a coast line is of more value than any theoretical attempt at calculation. The dynamical force exerted by a wave depends so much on local conditions that the theoretical results from open sea calculations are generally quite inadequate. For example, the effect of an ocean wave approaching a cliff barrier with deep water close in is very different to the effect of the same wave approaching a gradually sloping and shallow foreshore; or again, to its behaviour after constriction in the narrow entrance of a harbour mouth.

A great deal of study has been given to wave action, but it is admitted that there are so many variable functions that the *theory* of wave action is of little practical value. There are, however, several different types of sea whose action on marine works is well understood. These are (1) the travelling wave; (2) ground swell; (3) broken sea; (4) breaking sea; and (5) the oscillatory wave. These do not include the free tidal wave, but may be defined as follows:—

1. The travelling wave is one in which the particles of water have a definite forward movement (see Fig. 1).

2. A ground swell is the type of sea remaining after the wind drops. It is, of course, transmitted to areas beyond the storm zone, and is caused by the sub-surface velocity exceeding the surface velocity.

3. A broken sea occurs when the "range" is restricted from increasing; *i.e.*, the intervals between waves and the height of waves are small and cannot increase owing to shoals, currents, or changes of wind. This class of sea is not at all dangerous, from the engineer's point of view, unless superimposed on travelling waves.

4. A breaking sea occurs in shallow water when the wave collapses forward, owing to the arrest of its lower portion

on the foreshore or sea bed. The travelling wave, on reaching the coast, generally breaks in this fashion.

5. The oscillatory wave is one in which the particles of water have no forward movement. They are not damaging in effect as a rule.

The worst sea, from the engineer's point of view, is the breaking sea where the final onslaught of the wave is exactly on the alignment of any defensive works, sea-wall, mole, quay, etc.

FIG. 1.—Travelling Wave meeting an Obstruction.

Professor Arnold Hartley Gibson, D.Sc., in his paper entitled "Wave Impact on Engineering Structures," comes to some remarkable conclusions. These briefly indicate that whatever the face pressure of the wave may be (*i.e.*, exerted against a vertical plane), these pressures may be developed up to pressures 16 times as great if there are any open joints or fissures presented in that plane. This at once explains the necessity of wide spans in open structures and an unbroken surface in solid structures where such are exposed to the action of the sea.

Another important point to remember is that if, by reason

of artificial structures, an oncoming wave is thrown vertically upwards (see Fig. 2), a very intense downward pressure is caused by the subsequent descent of the water which, in exposed situations, may safely be taken as reaching 700 lbs. per sq. ft. This is a very important action, and the late Mr W. T. Douglass expressed the opinion in 1911 that probably 50 per cent. of sea-wall failures were due to this downward action destroying the surfacing behind the sea-wall proper.

FIG. 2.—Vertical Deflection of Oncoming Wave.

Both Mr A. E. Carey and Professor E. R. Matthews in their works give interesting records of damage by wave action, and cite extraordinary occurrences in respect of excessive weights moved, particularly in respect of damages to harbour works in the North of Scotland. One of the most remarkable effects noted by the author was on the South Breakwater, Dover, when under construction in 1906 (see Fig. 3). Concrete blocks were stacked by gantry cranes along the surface of the breakwater where it had been completed to formation level. In a very heavy storm in November of that year high waves broke over this breakwater and actually

shifted these blocks where stacked three deep, thus moving a column of concrete 120 tons in weight. These concrete blocks would have been swept right over the breakwater had they not encountered a quite unintentional barrier. Down the centre of the breakwater, which was 45 ft. wide, the contractor had laid a track of flat-bottomed rails temporarily secured by jag bolts grouted into the permanent work. The movement of these large stacks of blocks was arrested by these rails, and, although the track and rails were twisted, the fish-plates fortunately held and this prevented a disaster, which would have delayed the construction of the breakwater for

FIG. 3.—Admiralty Harbour, Dover. South Breakwater under Construction, 1906.

many months, as the blocks would have been thrown landwards against the contractor's falseworks, which they would undoubtedly have damaged. In a paper read before the Society of Arts in 1907 an original photograph of this occurrence was reproduced. The above case of wave action has been referred to at length, as it is probably the most reliable record of an excessive weight moved by the action of the sea well above high-water mark.

The late Mr C. F. Vernon-Harcourt deals with the theory of vertical oscillation in his book on "Harbours and Docks," but there is some doubt whether the generation of waves can be effected by wind without a definite forward motion being imparted to the particles of water. There is, however,

a valuable paragraph on the velocity of waves, and from Sir G. Airy's investigations, therein referred to, it would appear that the velocity of the wave, while increasing with its length (*i.e.*, from crest to crest), is also affected by the depth of the water, but that beyond a certain critical depth the velocity ceases to be affected thereby. There is some confusion of ideas as to what is meant by the "velocity" of waves. Such velocities have been observed by Mr Scott Russell and presumably represent not forward velocities but the period of undulation divided into the wave length, and this does not appear to be a measure of velocity at all. For example, a purely undulatory wave, which really does not exist by itself at all, would not be capable of doing any material damage to marine works by direct impact, and the engineer would then merely have to legislate for the simple hydrostatic pressures caused by the undulation (see Fig. 4). This is fairly evident by the breaking of waves against the side of a large ship *after* the aerial storm has abated. The real physical condition of most waves probably lies in a combination of both forward and oscillatory movements, while it may safely be said that the civil engineer has chiefly to deal with the wave of translation. Mr Vernon-Harcourt deals exhaustively with the subject, and perhaps unconsciously has thrown a good deal of light on the use of the term "range," and from the information he gives it is clear that there are waves of translation in a deep and open sea. There is some difficulty in directly measuring the force of the waves by means of a dynamometer which, by reason of the function it has to perform, has to be heavily constructed and the pressure recording plane designed to travel; this plane has considerable inertia, and the errors of observation are therefore obviously great. There is room for further experiment, and probably a purely hydraulic gauge could be designed.

Many text-books recite at length instances of the intense force of wave action, but these are so variable, and conditions so different, that they cannot really form the basis of any *calculation* of value. As, however, there is definite evidence of masonry masses of over 1,300 tons being moved by wave action, historical data is of value as indicating the importance of securing the most rigid structure possible and ensuring

perfect joints between the component parts of masonry or concrete work. These requirements call for the most careful specification and supervision, which is more than ever required in the case of monolithic concrete structures where a bad "bond" between old and new work is a real source of danger.

What may be classed as the free tidal wave is really of little practical importance and can best be regarded as the phenomenon which would occur on the normal diurnal tides

FIG. 4.—Oscillatory Wave meeting Obstruction.

provided there were no storm disturbances anywhere on the world's surface. Under such conditions the only physical effect would be a slow rise and fall in water level increasing through constrictions and in estuaries. Directly the first effect of land constriction is felt the oscillatory motion of a tidal wave becomes partially translatory, but more in the sense of a "current" than a wave. This statement is not universally true, as the tidal waves in certain cases become held back, and, suddenly advancing, cause a "bore wave," which is, of course, capable of damaging effect, as, for example, in the case of the River Severn. It must be made clear, however, that these tidal waves cause normal hydraulic

currents subject to the ordinary hydraulic laws, but they cause no damage beyond that resulting from the *scouring* currents produced (the few isolated cases of bore waves excepted).

On the cycle of tidal waves are superimposed the wind waves, and these comprise an entirely different series, and have definite damaging effects as previously cited. By virtue of the fact that wind waves are not simply oscillatory, their "range" is determined by the force of the wind, and the condition of the tidal waves on which they are superimposed. The depths of water also affect these waves, and they are the class of waves which may frequently be seen breaking on the coast line during storms. The after-storm wave is substantially of a different character, but it is the same class of wave altered only as to condition. The alteration is principally due to the fact that the sub-surface velocity has become higher than the surface velocity, creating what is known as "ground swell," previously referred to.

Beardmore deals in some detail with the subject of tidal waves in his book, and Mr W. F. Wheeler, contrary to the above view, considers that true tidal waves do "break" on the foreshore even when there is no wind, but the author is of opinion that this would not occur if still conditions existed simultaneously throughout the world. Mr Wheeler further considered the wind wave to be entirely undulatory, and he did not admit of any forward effect being produced until such a wave reached shoal water.

In recent years Mr Philip Brasher has successfully experimented with compressed air as a wave reducing agent, and several of his installations have been put down in the United States. He has ascertained that by releasing compressed air from a perforated pipe line laid on the sea bed, wave action is materially reduced. The author estimates from the data supplied him that roughly one brake horse-power per lineal foot of pipe line is required in a depth of 20 ft. of water. Where a narrow and dangerous harbour entrance requires protection, and where power is available, the Brasher system seems worth while experimenting with, but otherwise the expenditure of energy seems disproportionate to the advantages to be gained.

CHAPTER II

THE SALVAGE OF MARITIME WORKS[1]

Classification of Structures in Rivers and the Sea—Examples of Damage to Steel and Iron Structures—Damage to Reinforced Concrete Structures—Damage to Timber Structures—Examples of Damage by Storm and Collision to Harbour Works—Sea-Walls—Costs of Construction—Breaches in Embankments—Protection of Embankments—Costs of Maintaining River Walls—Summary of Selected Assessment Cases.

CIVIL engineers have frequently to deal with the reparation of marine structures damaged by storm or collision, or which are in disrepair from want of proper maintenance. When insurance questions are involved, their services are required more especially in connection with claims, the equitable settlement of which is the immediate concern of both owners and insurers. In cases where settlements are effected on a cash basis, the surveyor's task carries with it considerable responsibility, as the form and extent of the claims put forward by his clients are largely dependent on his advice. Should his estimates be referred to arbitration, or become the subject of an action in the High Courts, they are more than ever subject to criticism in detail. The preparation, therefore, of such estimates requires great care, and calls either for previous experience in assessment work of this character or, at least, access to data not always ready to hand.

The purpose of the present chapter is to give such references and data in general form as may be from time to time of assistance in the future.

The consideration of the subject necessitates a general

[1] Originally contributed to *Engineering*, and published in two sections, in issues dated 8th and 22nd January 1915, under the title "The Assessment of Maritime Works."

classification of works, and the following has been adopted :—

1. Piers, Wharves, Jetties, and Bridges constructed in—
   (*a*) Steel or iron
   (*b*) Reinforced concrete.
   (*c*) Timber.
2. Harbour Works, Docks, etc.
3. Sea and River Walls—
   (*a*) Constructed in masonry or concrete.
   (*b*) Earthen embankments.

The first of these three divisions covers by far the largest field, including, as it does, in addition to many important structures owned by corporations or other public bodies, the numerous privately owned wharves and jetties which abound in the confined navigable waters of the principal English rivers. Such works are, of course, peculiarly liable to damage from passing shipping, or to accidents arising from errors of judgment in the berthing of vessels alongside.

Steel and Iron Structures

Experience goes to show that the extent of damage to steel or iron structures is usually less, and the assessment easier, than in the case of similar works constructed in reinforced concrete. Where steel piles are concerned there is generally little danger of actual fracture of the pile due to collision, failure usually taking place at riveted or bolted joints between the pile heads and superstructure. When a steel pile in the immediate vicinity of the damaged portion of a steel structure is observed to remain true to position and line above low-water level, it is practically certain that no *fracture* has occurred below ; this cannot be said to be equally true of reinforced concrete.

A steel or iron structure lends itself readily to the erection of a contractor's temporary staging, and the repair work is usually rapid. On the other hand, very careful attention must be paid to the general condition of the work. Unfortunately, it is only too common to find steel or wrought-iron piers sadly neglected, and therefore corroded to an

appreciable extent, "condition" being, of course, a most important consideration where questions of valuation are concerned.

It must always be borne in mind when dealing with collision cases that although a structure may be old and in bad repair, the owner is entitled to its reinstatement after damage in such a manner that it is left in as good condition as before the accident. It will be obvious that, in the case of old steelwork, any attempt at patching piecemeal cannot be said to fulfil this requirement, and, as a result, the owner is usually successful in securing repair work which gives an appreciable increase of strength to the structure, the structure, as a whole, being left in better condition than before the collision took place. In many respects reparation in steel-work is an easier operation even than in timber, though necessarily not quite so rapid.

Under the heading of "Steel and Iron Structures" may be cited the following instances:—

(i.) S.S. *Wandle* in collision with Wandsworth Bridge, June 1912.

(ii.) Reparation of piling, Clacton Pier, 1914.

(iii.) Sailing ship, *General Foy*, in collision with coaling tip, Port Talbot, November 1910.

(iv.) S.S. *White Heather* in collision with swing bridge at Poole, March 1908.

(v.) Barge, *Bassildon*, in collision with Southend Pier, December 1913.

The above are dealt with *seriatim* below:—

(i.) Wandsworth Bridge was completed in 1873, to the designs of the late Mr J. H. Tolmé. The bridge consists of wrought-iron lattice girders in continuous form, carried on piers, each consisting of two wrought-iron cylinders, 7 ft. 6 in. in diameter, filled with mass concrete, and at 30 ft. transverse centres. There are five spans to the bridge, the central spans being 134 ft. each. The cylinders in each pier are attached together at the top by a small transverse lattice girder, riveted direct to the shells of the cylinders, which at their heads are only $\frac{5}{16}$ in. thick. The S.S. *Wandle* struck one of the mid-stream columns with her port bow, and displaced it 5 ft. 6 in. to the south and 1 ft. 6 in. to the west. The cylinder was

found to be fractured 46 ft. below the lower boom of the main lattice girder, leaving a length of 18 ft. standing vertical in the clay below the river bed—*i.e.*, to foundation level. Examination of the main girders showed no adequate provision to have been made against their lateral displacement by possible collision from river craft. The piers of the bridge were and still are unprovided with dolphins, or any form of protection against collision, and it is remarkable that no accident had previously occurred during the forty years of the bridge's existence.

Repairs were effected by the London County Council, and the laborious method was employed of sinking a cast-iron caisson, 16 ft. in diameter, through the Thames ballast into the clay to a level of -33 Ordnance datum, and carrying same up above Trinity high water to a level of 16 Ordnance datum. The wrecked column was ultimately plumbed inside the caisson and the fracture repaired. To protect the reconstruction works from collision, a temporary diamond-shaped "island" was constructed in the river, comprising twenty-eight timber piles provided with floating booms. The photograph (Fig. 5) shows the bridge under reconstruction.

Calculation showed the total weight of the column damaged to be 175 tons, and the weight of the displaced portion to be 122 tons. The column was struck 20 ft. below the under side of the main girder, and considerations as to the strength of the column led to the definite conclusion that had the collision been caused by a barge of only 100 tons displacement, drifting on a 2½-knot tide, a similar result would have occurred. Fig. 6 illustrates in diagrammatic form the stability of the column, from which the force of the blow necessary to overcome its stability is apparent.

The ultimate cost of reparation was £4,247, and would have been considerably more under post-war conditions.

This is one of the few and, consequently, interesting cases in which the force of impact could be ascertained within reasonable limits of accuracy. An important deduction from this collision is the great disparity existing between the cost of repairing such a column as against the initial cost of construction. The main girders had to be carried during reconstruction on two temporary stagings erected on either

side of the damaged column in the manner indicated in Fig. 7. It is an open question whether the method of repair adopted was the cheapest that could be undertaken, and at

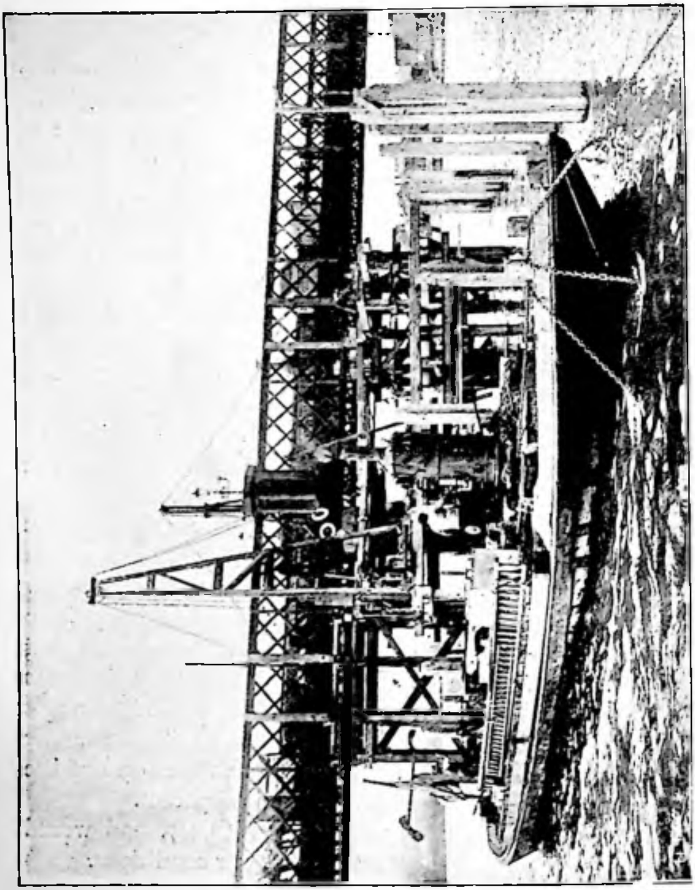


Fig. 5.—Wandsworth Bridge under Repair, 1912.

the time, on this point, a considerable difference of opinion existed between the experts called in to represent the parties concerned.

(ii.) The work at Clacton Pier is interesting as an illustration of certain advantages which cast iron and steel

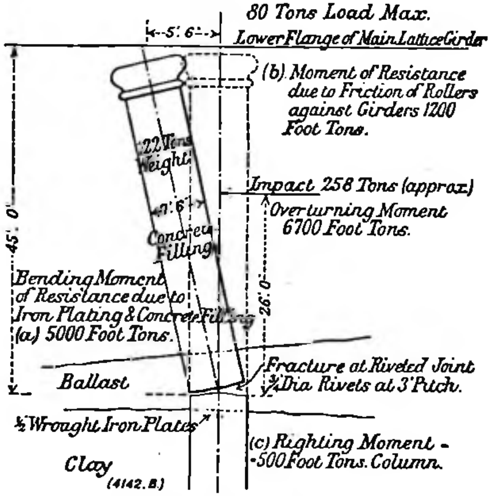


FIG. 6.—Stresses on Column of Wandsworth Bridge.

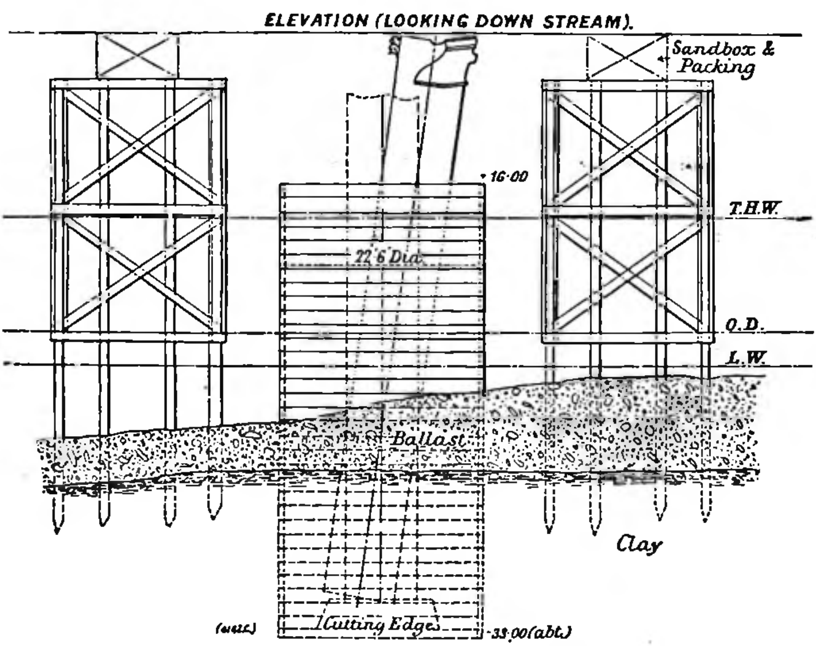


FIG. 7.—The Repair of Wandsworth Bridge, 1912.

reparation possess over timber. Fig. 8 illustrates a normal cross section of the pier head. The original structure comprised the 12 in. by 12 in. raking piles of Memel fir shown in the centre of the bay. The pier head was subsequently widened to the extent shown on the cross section, and a pavilion constructed thereon, of which the outline is indicated in the figure. Certain steel piles were driven to carry the stanchion loads of this pavilion, but no provision was made at the time to carry the floor load in the event of decay in the piles beneath. The raking piles referred to were driven

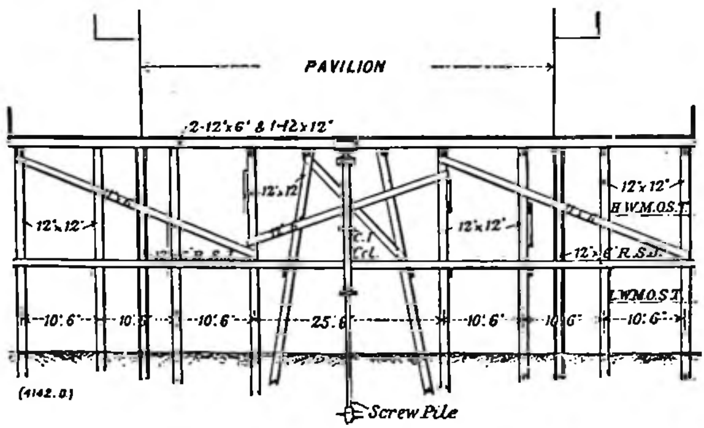


FIG. 8.—Cross Section, Clacton Pier Head.

in 1878, and at an inspection made in February 1914 they were found to be seriously decayed at low-water level.

In advising the Coast Development Corporation, as owners of the pier, it was necessary to recommend resort to screw piling with superimposed cast-iron columns to carry a steel girder longitudinally down the centre line of the pier head. Any idea of employing reinforced concrete piles had to be abandoned, because of the difficulty of driving and manipulating in the confined space below the deck. For the same reason the employment of timber piles was impossible, as the pavilion was in constant use and the deck could not therefore be lifted to secure the necessary headroom for driving. With cast-iron screw piles, however, it

was found to be practicable to employ a screwing length of 16 ft., and to include a screw with a blade of 3 ft. 6 in. diameter; above the screwing length each column was carried upwards in two short 8 ft. length sections, flanged and bolted together. The first section of this work was completed in July 1914, and the cost worked out to about £107 per pile, including girder work and saddles to carry the pitch-pine deck bearers.

As screw piling is frequently resorted to in reparation work, it may be of assistance to indicate the best form of blade to employ. Figs. 9 and 10 represent a common form of blade with two convolutions. This form possesses the advantage of screwing absolutely true in most foundations, but if used in a clay bed or gravel where boulders are likely to occur, there is no escape upwards for the latter, and it then becomes impossible to screw the pile to its specified depth, while the penetration reached may be insufficient to support the working load. Figs. 11 and 12 illustrate the type of blade employed at Clacton. This blade has only one convolution, and, as shown in plan, the cutting edge is cleared well away and the heel "backed off." This gives great facility in screwing, and is the form to be recommended for most classes of reparation work. It possesses, however, one disadvantage, and that is, that the screwing has to be carefully watched, as there is a slight tendency for the single-bladed screw to "creep." A further important advantage possessed by screw piling is the small deck area occupied by the driving plant, and the fact that by the judicious use of snatch blocks between the capstan and winch, several piles can be screwed without moving the plant. The work at Clacton was carried out using a deck space of only 25 ft. by 25 ft., and the normal traffic of the pier was not interfered with in any way.

(iii.) Cases of vessels colliding in dock with coaling tips are of common occurrence. The sailing ship, *General Foy*, after leaving dry dock, in coming alongside at Port Talbot on 22nd November 1910, fouled the upper diagonal leg of the tip, which in this case was 64 ft. long at a point 24 ft. above water level, buckling the leg to the extent of about 8 in. The leg was constructed of mild-steel plates and angles in four sections, each 16 ft. long, the face plates being 9 in. wide

and the side plates 12 in. wide respectively. Reconstruction necessitated the removal of the entire leg and the replacement of the damaged section by a new one. This accident may be taken as a typical example of damage to a steel strut caused by a colliding vessel. The Dock Company ultimately accepted £130 in settlement, and on this basis the pre-war

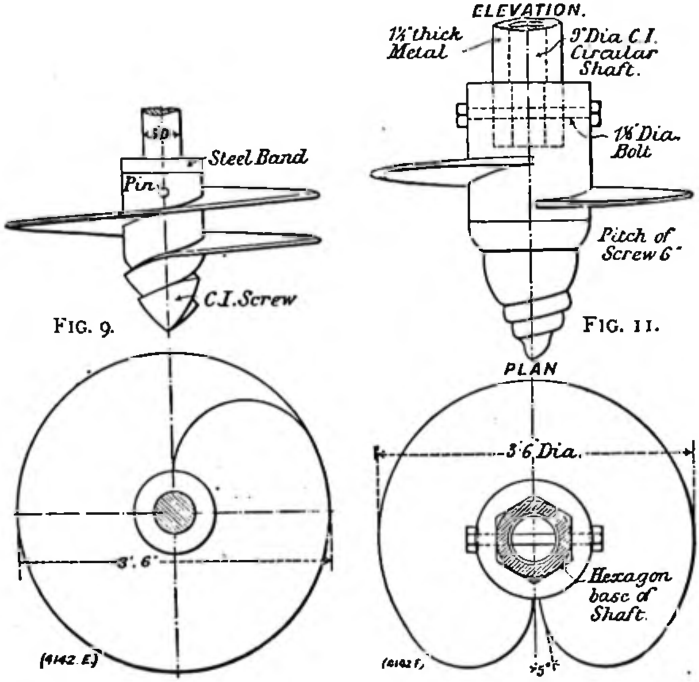


FIG. 9. FIG. 11.

FIG. 10. FIG. 12.

The Strengthening of Clacton Pier, 1914.

reconstruction cost may be regarded as working out at £2. 0s. 8d. per foot run. The damage extended to the angle-iron bracings attached to the leg, and therefore the above figure is useful as one including the making good of such attachments and the erection of the necessary temporary staging for effecting repairs.

(iv.) The swing bridge at Poole Harbour was run into by the S.S. *White Heather* on 6th March 1908, and suffered

extensive damage. The bridge was constructed in 1885, of iron, with protective piling and dolphins in timber. The reparation work in this case included principally the screwing of eight new piles, constructed in mild steel and cast iron, the withdrawal and redriving of three greenheart 14 in. by 14 in. by 40 ft. piles in one of the dolphins; and five pitch pine 12 in. by 13 in. by 40 ft. fender piles in the bridge foundations; also the replacement of certain mild-steel joists and gusset plates of the bridge girders. The reparation work outlined above was executed at a cost of £1,050, after a sum of approximately £570 had been spent on emergency work, temporary repairs, surveys, and inspections. The accident is of special interest, for the following two reasons:—

In the first place, it raises a point of considerable importance in assessment. The design of the main piles carrying the entablature of the swinging span was old-fashioned, consisting of mild-steel plates $\frac{3}{8}$ in. thick, forming a riveted column 12 in. in diameter, fitted with cast-iron flanges, to enable each 8 ft. section to be bolted to the next, the whole column being filled inside with 6 to 1 mass concrete. In reconstruction the same design was followed for the sake of uniformity and in accordance with the requirements of the bridge owners. Such columns are nowadays relatively costly and present difficulties in connecting with the screwing section for driving purposes. In any such bridge to-day doubtless cast-iron screwing lengths and columns would have been employed throughout, and their replacement would have been an easier and less costly undertaking. It may be taken as a general rule, therefore, that the earlier the original date of construction in steelwork or iron, the higher will be the cost for replacement of each unit.

In the second place, one curious result of the collision was the fracture of a 3 in. diameter spigot and socket cast-iron gas main conveying the gas supply from Poole across the channel to Hamworthy. The gas main was carried across on the bed of the channel, protected by encasement in hollow timbers attached to the structure of the bridge. Efforts to effect emergency repairs to this main failed, and a flexible 3 in. hose was carried across, which had to be lowered to the bed of the channel to allow the passage of each vessel.

As subsequently it was found impracticable to lay the new permanent main on the old site, a new main was provided clear of the bridge, necessitating, in order to establish it, considerable diving and the construction of two additional temporary dolphins in mid-channel, from which the main could be lowered to its new position. The ultimate cost of this operation, in addition to the contract price for bridge reconstruction quoted above, amounted to £417.

(v.) On 4th December 1913 the sailing barge *Bassildon*, 43 tons, ran into Southend Pier, fracturing four lengths of cast-iron screw piles, each 11 ft. long and 8½ in. in diameter and ¾ in. thick. This accident was accompanied by a corresponding amount of damage to tie-rods, stanchions, railings, etc. There was no actual rescrewing of piles necessary in this case, the lower sections with flanges being left intact with the exception of one case, in which a new flange was provided and the shaken pile bedded in concrete.

Reinforced Concrete Structures

There is little doubt that engineers experience some difficulty when called on to assess damage to reinforced concrete structures. The repair work in these cases is slow and tedious, and the extent of the work necessitated is frequently difficult to ascertain until the demolition of all stressed members is complete. The following cases of damage to reinforced concrete jetties are amongst those which have come directly inside the author's experience. It is to be regretted, however, that the information in regard to reinforced concrete structures which it is possible to give is limited, owing to the difficulty in extracting details from wharfingers as to the actual costs of reparation, which costs, if properly appreciated, would doubtless lead to an increase in insurance rates with the underwriters. The cases quoted below are of interest:—

(i.) S.S. *Goorkha* (Union Castle Line) in collision with Dagenham Dock, April 1907.

(ii.) S.S. *Florence* and barge *Fred* (1,057) in collision with jetty, Halfway Reach, River Thames, February 1911.

(iii.) Claims in connection with jetty at Gravesend, 1911.

(i.) In the first case, the S.S. *Goorkha* is reported to have been deliberately put ashore in order to avoid collision with another craft. The dock in question included a reinforced concrete jetty, forming a valuable riverside property, with considerable berthage in deep water. It was equipped at the time with four electric hoists and two rail access tracks down the approach. The vessel struck the wharf stem on, causing the severance of one bay, and the severe stressing of two adjacent bays. One of the access tracks was rendered useless, and the working of the jetty thereby largely interfered with. The damage on first inspection did not appear great, but a further survey of the reinforced concrete struts and bracings in the two adjacent bays referred to disclosed the existence of several manifestly new "hair-line" cracks. It is clear that when the existence of such a crack running obliquely across and round the strut is discovered, no value can any longer be attached to the compressive resistance of the member. There was, therefore, in this case no resource left but to recommend the removal of the entire strut, since the alternative of a reinforced concrete sleeve over the fracture—admittedly an unsightly and makeshift arrangement—did not appeal to the owners of the jetty. The estimated cost of reconstruction in 1907 was in this case approximately £1,600. It was strongly to be inferred in this instance that, had the jetty been constructed of steel or timber, with the joints of all members readily accessible, the estimated cost of reparation would have been considerably less. In a similar instance of a collision between a steamer and a reinforced concrete jetty, which occurred in 1912, a diver's inspection showed the development of considerable cracks in the concrete columns below low water, and at a considerable distance from the actual point of impact. In the case of damage to jetties of this class, a diver's inspection is always desirable; frequently, no indication of straining or fracture is apparent above low water, whereas serious damage to the same member may exist below.

(ii.) In the second case quoted above, a small steamer proceeding down stream fouled a barge lying alongside a light reinforced concrete jetty constructed in open piling. As is common in structures of this type, the piles were

driven to a point about 10 ft. below deck level, and their reinforcement connected direct with that of the columns, walings, and bracings of the superstructure. The freeboard of the barge, at the time of the accident, did not reach the level of the lower walings, and the force of impact fractured the pile at its head; the pile was carried away bodily, leaving the superstructure standing intact. The deck was thus rendered unsafe to carry its normal load across the enlarged span. This accident is another typical case of the difficulties of repair in reinforced concrete, as reparation had finally to be effected by driving another pile alongside the site of the old one, it being, of course, impracticable to remove any members of the superstructure, which would have been an easy operation had the jetty been constructed in timber or steel.

(iii.) The third case cited as an example of damage to reinforced concrete structures is not one of damage caused by a colliding vessel, but is an instance of repairs to work below low water, which work was found, as a result of a diver's inspection, to be faulty. The jetty in question is at Gravesend, and of considerable extent, being carried partly on 5 ft. diameter reinforced concrete columns, and partly on 4 ft. diameter reinforced concrete columns. The work was commenced in 1909, and the faulty condition of the columns was not discovered until the works had reached an advanced stage. The author was asked to advise in 1911 in reference to the claims of the contractors for extra work involved in the reparation of faulty columns. This reparation was effected by means of reinforced concrete sleeves, constructed *in situ* round the faulty columns inside annular mild-steel casings. The cost of this extra work was considerable, representing over 8 per cent. on the total cost of construction. Under the terms of the original specification it does not appear possible that satisfactory columns could have been secured; the specification in this respect was substantially the same as that for a jetty previously constructed in the Thames estuary. In substance, the specification required the contractor first to drive two reinforced concrete piles, and, secondly, to sink round them a temporary cylindrical mild-steel casing, to be properly bedded from the outside by

divers. The reinforcement of the cylinder had then to be placed in position, and mass concrete deposited through chutes in the top and rammed. In many cases, on removal of the steel mould, these columns were found to be honeycombed, with the reinforcement exposed. In the parallel case referred to, by wisely amending the specification of the Licensing Company in respect of the procedure required of the contractor, the engineer found it possible to obtain sound columns. To effect this, however, he found it necessary, first, to sink the steel casing; secondly, to send the diver down inside the cylinder to make a tight joint with the river bed from the inside, then to drive the piles in their proper positions inside the cylinder, and finally to deposit the mass concrete, and ram in the usual manner. This operation was materially different to that required in the original specification. As jetties of this type are now[1] being freely adopted by wharfingers, this is an important point to bear in mind when dealing with the reconstruction of reinforced concrete columns.

Timber Structures

Dealing next with timber structures, their most noticeable feature is the great rapidity with which repair work can be carried out, either in the case of normal maintenance, or as repairs to damage caused by collision or storm. The great disadvantage in the employment of timber is, of course, the question of decay. With, however, the proper specification of creosoting, Burnettising, or the use of other acknowledged and well-tried preservatives, the ordinary life of timber structures in this country can be extended to about twenty-five years, and if their maintenance is properly undertaken, and the structure not neglected, its life may be indefinitely prolonged. It is a type of construction admirably adapted to wharves and jetties in confined navigable waters. The following collision cases are of interest:—

(i.) S.S. *Harrovian* in collision with jetty, Long Reach, River Thames, 1908.

(ii.) Damage to jetty, Purfleet, 1911.

[1] 1922.

(iii.) S.S. *Narva* in collision with tidal dock, Surrey Commercial Docks, 1907.

(iv.) S.S. *Ryhope* in collision with jetty, Tilbury, 1914.

(i.) The first example illustrates the flexibility of timber structures when subjected to the shock of a colliding vessel. The S.S. *Harrovian* was proceeding down river, and when off the jetty swung inwards, due to the failure of her steering gear. The vessel ran ashore, stem on to the river bank up stream of the jetty head, and her stern swung round on the ebb tide, colliding with the diamond end of the T-head. This jetty was a heavily-timbered structure, and three bays at the western end of the jetty head were severely stressed, the centre line of the jetty at its extremity being set over to the extent of 24 in. at deck level. A cash settlement in respect of the damaged portion of the jetty was effected for the sum of £1,100. Had the jetty in question been constructed in reinforced concrete, there is little doubt that the damage caused would have been much greater in extent, the piles would undoubtedly have been fractured, and the cost of repair, therefore, much higher; furthermore, in all probability the jetty would have been thrown into disuse for a lengthy period, whereas in this particular case there was not even any interruption to traffic.

(ii.) In the second case the damage was of a more serious nature. The jetty in question is constructed at right angles to the river bank, being provided with a T-head in about 20 ft. of water at low tide, and as the photograph (Fig. 13), taken shortly after the accident, shows, the colliding steamer passed right through the jetty approach. The main piles of the jetty were constructed in Jarrah, and the remaining timbering in pitch pine. As shown in Figs. 14 and 15, twelve main piles were broken, certain moorings and buoys were cut adrift, and five bays of 15 ft. span each were demolished, four similar bays being severely stressed. In spite of the serious nature of this damage, the complete reparation of the jetty was effected within a comparatively short period; the reparation contract was let on 31st October 1911, and the work was sufficiently complete in March 1912 to allow the business of the wharfingers to be resumed, thus occupying a period of only four months, a good example

of the rapidity with which repair work to timber jetties can be effected even in the winter.

(iii.) In the third case the S.S. *Narva* was lying in a

FIG. 13.—Damage to Timber Jetty at Purfleet.

small tidal dock, and being aground at low water, became "mud-sucked" on a rising tide. On freeing herself from the river bed she snapped her cables and surged against the timber wharf face. The extent of the damage was, however, slight, being estimated at only £50; had the whar

face been constructed in masonry, concrete, or reinforced concrete, the damage would certainly have been greater both to wharf and ship.

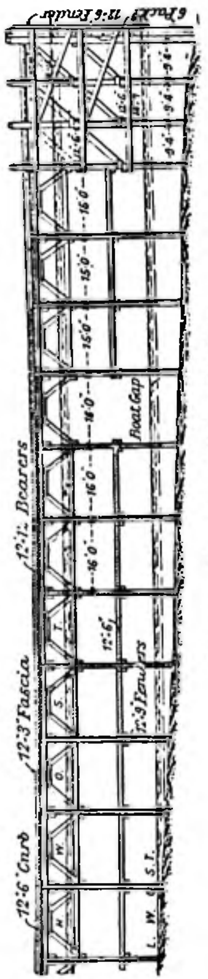


FIG. 14.—Elevation of Jetty, Purfleet.

Members thus were broken, otherwise damaged, or removed by collision. Deviation from plumb of piles is shown by arrows thus (deviation in 300 lg.th) Deck Levels over pile heads shown thus Arbitrary Datum. Nomenclature of piles shown thus.

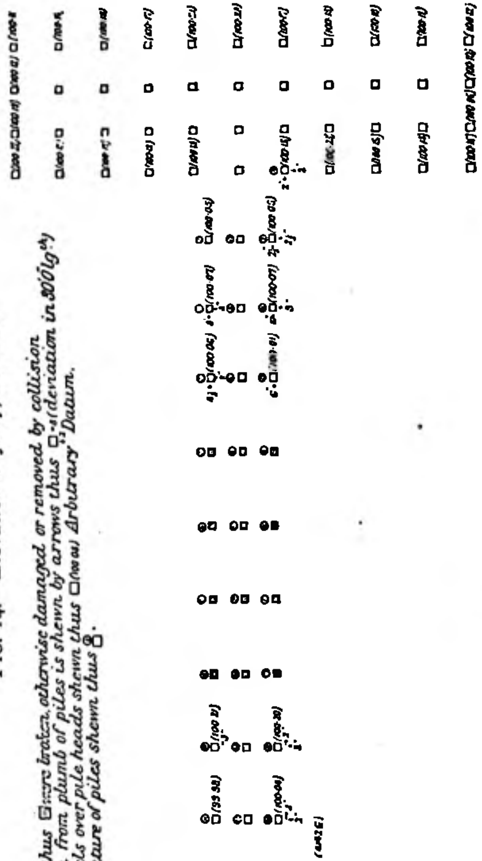


FIG. 15.—Chart of Damage, Purfleet.

(iv.) The last case is one of the few in which reparation, although to a timber structure, was a difficult question. The accident occurred while the vessel was manœuvring to her berth. The port bow fouled one of the main piles of the

jetty face. Under normal circumstances, the fractured pile could either have been withdrawn or scarfed with ease; in this case, however, the jetty was very stiffly braced and heavily timbered, and had been considerably strengthened since its original construction. The main piles were of extreme length, owing to the bad nature of the ground encountered during driving. These main piles were in some cases scarfed at two levels, a single pile occasionally possessing an aggregate length of 140 ft. The withdrawal of the pile was impracticable, or, indeed, its removal to the first scarfing, since the head of the pile itself formed a mooring bollard in constant use, without which vessels could not berth alongside the jetty. At an inspection made on behalf of the insurance company, it was ultimately decided to carry two 14 in. by 7 in. timbers alongside the damaged pile, from deck to low-water level, and to bolt the same thereto at intervals of 4 ft., which course was finally adopted. From a purely engineering point of view, however, such a repair is not entirely satisfactory, although there is little doubt that in this case the reconstructed pile was as strong, if not stronger, than the original. In the case of a steel column, the damaged sections could have been removed intact, and the difficulty in respect of the mooring bollard would probably not have occurred; indeed, the expedient of using the pile head for this purpose is not common, either with steel or reinforced concrete structures, nor, in view of the nature of this accident, would it appear to be a desirable design in timber.

Having cited a few examples falling within the first division of the classification adopted, it may be of use generally to summarise those points which experience has shown to be of importance to the engineer or surveyor in dealing with estimates for reparation. It is generally dangerous to prophesy in haste the actual expense likely to be incurred in effecting repairs to damaged piers, wharves, jetties, or bridges. If the damage is of any extent, a detailed survey should be made at once, above water, and a diver's inspection below. Two schedules of quantities should be prepared, the one dealing with the removal of old or damaged work, and the other with new work in the usual form of a contract

schedule. Experience has shown this to be a sound procedure, as it lessens the likelihood of extra claims being put forward by the contractor during construction, or of any subsequent misunderstanding on his part as to the true nature of the work—sometimes a cause of friction between engineer and contractor in reparation contracts. These schedules further enable the surveyor to price the work in detail, and thus to arrive at a close estimate as to the total cost of effecting repairs. Where possible, in addition to the working drawing, there should also be incorporated in the contract documents outline diagrams attached to an agreed schedule of nomenclature, in which every member of the structure receives a designating letter or number; such classification can then be adopted in the schedules of quantities. There are times, however, when a rough estimate as to cost has to be given on the spot. This generally applies to cases where the structure is left in a dangerous condition through collision, and works have to be undertaken immediately, quite apart from any questions of liability. Such an assessment has generally to be made and agreed between the technical advisers before any contract can be let, which latter is then usually on a cost and profit basis, and the engineer's estimates, therefore, are of paramount importance, being unsupported by any contractor's tenders.

In dealing with cases of this character at short notice, the accompanying table dealing with pre-war costs (Table I.) may be of interest, though it would be unwise to suggest that it necessarily affords any great assistance in dealing with the assessment of specific damages to any existing structure; it should rather be regarded as giving in very general terms some indications of the unit prices likely to be involved in reparation contracts of the character under discussion. Such information, however, as is given in the table has been abstracted from actual contract prices or from schedules prepared by the surveyors concerned. If the unit prices given are to be taken as the basis of a rough estimate, percentages varying from 10 to 40 per cent. must be added to any total so reached, in order to arrive at anything like even a rough approximation of the cost of reparation. Within the limits of such a table it is not possible to give dimensions

TABLE I.—UNIT PRICES OF REPARATION CONTRACTS (PRE-WAR COSTS)

Structure.	Description of Works.	Piles.		Struts and Walings.	Diagonals and Bracings.	Deck.		Columns.		Fendering.	
		Section.	Price per Lineal Foot Driven.			Thickness.	Price per Square Yard.	Section.	Price per Lineal Foot.	Section and Material.	Price.
							£. s. d.		£. s. d.		
1. Timber: Jarrah and pitch pine, 1911	Jetty approach	14 in. by 14 in.	10s. 6d. including withdrawal of old pile stumps	4s. 4d. per cub. ft.	4s. 4d. per cub. ft.	3 in. pitch pine	0 7 6	...	...	Elm; 12 in. by 8 in.	7s. per cub. ft.
2. Steel, 1908 -	Passenger steamer landing-stage	...	...	2s. 9d. per lineal foot	5s. 7d. per lineal foot	Wrought-iron gratings	0 11 1	...	...	...	...
3. Timber-work on (2) above pitch pine	Passenger steamer landing-stage	...	...	7s. 10d. per cub. ft., including allowance for steel cleats	...	...	...	...	...	...	...
4. Cast iron and steel	Pier head	...	...	£2. 8s. per lineal foot, compound mild-steel girders, 15 ft. span and 14 in. deep	...	...	...	9 in. cast-iron columns, complete with screwing section, and 3 ft. 6 in. diameter screw	2 1 2	...	...
5. Mild steel -	Leg of coal tip	...	...	...	...	...	...	Face plates, 9 in. wide. Side plates, 12 in. wide	2 0 8	...	...
6. Cast iron -	Pier	Upper sections of cast-iron screw piles, including damage to tie-rods and adjacent railings	£ s. d. 2 3 3	...	...	...	...	...	...	...	...
7. Reinforced concrete, 1907 -	Pier head	Octagonal, 15 in. across flats	0 10 0	5s. 6d. per cub. ft.	5s. 6d. per cub. ft.	6 in. 1½-in. granolithic paving	0 15 0 0 7 0	5 ft. 6 in. diameter surrounding piles	1 11 0	Elm; 12 in. by 8 in.	12s. per cub. ft.

or description of the members priced in detail, but it so happens that in jetties, piers, and wharves in the tidal waters of Great Britain there is to some degree a certain similarity in design which alone makes any attempt at unit tabulation possible.

2. *Harbour Works, Docks, etc.*—Stress of weather is, without doubt, chiefly responsible for damage to harbour works, if the term "harbour works" is taken to include breakwaters, moles, training works, etc., and not works forming, or supplementary to, floating basins and graving docks.

The Government works at Port Patrick[1] were finally abandoned in 1870, after their construction had been commenced in 1863, owing to the constantly recurring damage caused to the works by heavy seas sweeping over the breakwaters; 500 lineal ft. of the south pier and 300 lineal ft. of the north pier were destroyed in this manner. The piers were constructed with rubble hearting, protected by ashlar masonry faces. The cost of the south pier alone and its adjacent works was approximately £60,000, and the port is now entirely derelict. The fishing port of Mevagissey, in Cornwall, suffered severely in 1891 from stress of weather, and involved the construction of new piers at a cost of £32,000; while the harbour works at Wick have suffered severely from time to time by the onslaught of the sea.

The above are examples of actual damage to artificial works projected into the sea. There is another and equally serious aspect in connection with works of this character, viz., the liability to silting up of the entrance channels, or the obliteration of the works by rapid accretion on the adjacent coast line. Where this phenomenon is slow, it is of academic interest only, and the subject has been extensively dealt with in technical literature; the ancient or partially derelict harbours of Sandwich, Pembrey, Lympne, Rye, etc., are examples of this class. When, however, this phenomenon is rapid, it may well come within the purview of the engineer and information is frequently called for as to the exact nature and estimated cost of works (if any) which it is desirable to undertake.

[1] A. E. Carey, M.Inst.C.E., on "Fishery Harbours and Harbours of Refuge." St Petersburg Navigation Congress, 1908.

Fig. 16 is a plan of Burry Port Harbour, South Wales, where difficulties were experienced with the sanding up of the entrance channel in December 1912. Marine surveys of considerable extent were undertaken by the author during the year 1913 and the early part of 1914. From many observed levels the author has taken one case as an example. The line A B, shown in Fig. 16, represents in plan the location of a cross section on the foreshore, originally taken in February 1913, and repeated in July 1913, November 1913,

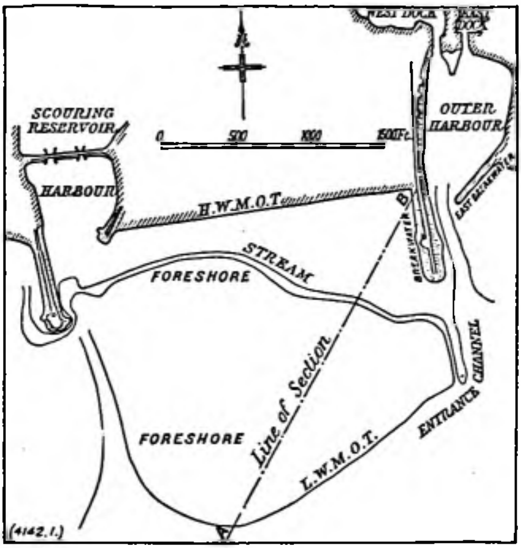


FIG. 16.—Plan of Burry Port Harbour, South Wales.

February 1914, and May 1914. Fig. 17 shows the profile of these observed levels.

The data obtained from the five sections taken show that there was an average accretion on the foreshore representing a rise in level of about 4 ft. over a length of 1,600 ft. This fact, combined with a series of similar sections taken to the east and west of the example quoted, made it possible to estimate the rate of accretion, which worked out to about 50,000 cub. yds. a year. Dredging would in this case have been of doubtful utility, and, at any rate, would have been a

constant maintenance charge, amounting to £1,350 per annum, on a basis of 6d. per cub. yd. dredged (a pre-war estimate). In the case of the harbour works in question this would have been altogether too heavy a burden, and the author, therefore, advised an expenditure on permanent works at an estimated cost of £15,000, which, capitalised at 5 per cent., represented the reduced figure of £750 per annum. The blockage of the entrance channel by sand, after the first instance, was threatened again in February 1913 and in February 1914, while from levels, tidal and current observations taken in the vicinity, there was little doubt that if

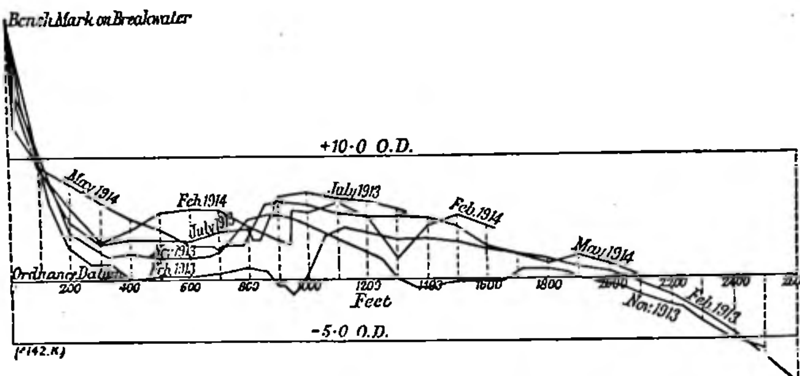


FIG. 17.—Profile of Burry Port Foreshore.

remedial works were not undertaken the harbour would be choked with sand, which, in fact, later occurred.

An instance, and perhaps an exceptional one, of serious damage occurring to harbour works during construction, was that of the collision between the S.S. *Olaus Ollsen* and the falseworks of the South Breakwater, Dover Admiralty Harbour, in 1906. These falseworks consisted essentially of gantries on either side of the breakwater site; they were constructed in deep water in 16 in. by 16 in. blue-gum piles. Each pile cluster consisted of six piles braced together at the top, with the centres of the clusters 55 ft. apart longitudinally and 100 ft. laterally. The gantry decks, Goliath, and service roads were carried on a system of mild-steel lattice girders, and the weight of the plant was considerable. The total weight of

the Goliath cranes was 350 tons each, and they were each capable of lifting and transporting 40 ton blocks. The vessel collided with the seawards gantry near its western extremity. The actual impact of the collision was apparently not very great, but the level at which the ship struck happened to approximate to that at which the main piles of the gantry were scarfed. The scarfing was effected by the usual form of joint, strengthened by means of two mild-steel plates bolted through the piles, and carried for some distance above and below the joint itself. The plates corresponded in alignment with the direction of the gantries, and were therefore parallel to the centre line of the breakwater (*i.e.*, at right angles to the probable direction of impact in collision), and in the position of least resistance to bending. Calculation indicated that had the plates been placed at right angles to their actual position, their resistance to bending would have been at least twenty times as great. Three bays of the seawards gantry were destroyed by this collision, and the plant and girders thrown to the sea bed. The weight of the wrecked plant at its northern end was thrown against the landwards gantry, as shown in the photograph (Fig. 18), thereby fracturing many of the piles close to the sea bed.

The wreckage was ultimately entirely dropped to the sea bed by severing the blue-gum piles at ground level, for which purpose blasting gelatine was used, the charges being tamped to the piles by divers and discharged by means of electrical detonators. Great caution had to be used in securing the complete severance of the wreckage from the sound portions of the gantry without damaging the latter. The wreckage was ultimately removed by cutting the principal members with dynamite charges and lifting the component parts from the sea bed. The claim against the insurers in this case exceeded the sum of £27,000, or approximately £186 per lineal ft. of the falseworks damaged. The greater part of the cost of reparation of these falseworks devolved ultimately upon the insurance companies, as the amount in question was greatly in excess of the statutory liability of the vessel. It is interesting to note that the maximum amount which can be recovered from any vessel for damage caused by collision is £8 per ton on her gross register, except

in cases where her liability is specially determined by Act of Parliament.

With reference to damage caused to docks, this is to be found principally in connection with caissons and dock gates;

FIG. 18.—Wreckage of Plant at South Breakwater, Dover Harbour, 1906.

such damage is of frequent occurrence, and occasionally involves large amounts. One of the most interesting cases with which the author was connected was an accident at Jarrow-on-Tyne. The collision in this case was the result of the launch of the S.S. *Patris*, 2,500 tons displacement, on

23rd December 1908. This vessel was launched from the yard of the Northumberland Shipping Company, Howden. After taking the water, the vessel got out of control owing to the severance of a cable and the consequent fracture of a drag-box shackle. Opposite the site of the launch, and on the other side of the River Tyne, is one of the dry docks owned by the Mercantile Dry Dock Company, and the S.S. *Patris*, as a result of the mishap, fouled the gates of this dock stern on, penetrating some distance into the dock. It was a fortunate circumstance that the dock had been flooded some ten minutes before the accident took place. The accident involved the complete demolition of one leaf of the dock gate and the severe straining of the other leaf; damage also resulted to the greenheart sill of the dock, and slight damage to the stern of a ship lying therein. The estimated extent of damage to works was £7,500, and a further claim substantially in excess of this figure was put forward in respect of consequential damages, loss of trade, etc. In the amount covering damage to works had to be included the cost of establishing, maintaining, and removing a timber coffer-dam erected outside the dock entrance, inside which the gates were re-erected. A general plan and cross section of the coffer-dam in question are shown in Figs. 19 and 20, the designs being prepared by Col. J. Mitchell Moncrieff, M.Inst.C.E.

As a contrast to this case of extensive damage may be cited the collision between the S.S. *Teeswood* and the west leaf of the inner gates of the lock of King's Dock, Swansea, in 1913. Being late for the tide, the vessel was coming down the dock to tie up near the entrance, and, overrunning her mark, struck the dock gates. Owing to the *Teeswood* having a cut-water stem, she crushed the deckway, brackets, and the steel plates of the dock gate nearly down to water-line; the damage, however, was not, as in the previous case, of sufficient extent to render the gates useless, and traffic was therefore not seriously interrupted.

3. *Sea and River Walls.*—Sea-walls in masonry and concrete of the type met with at seaside resorts are subject to failure in two ways, either from pressure from behind or

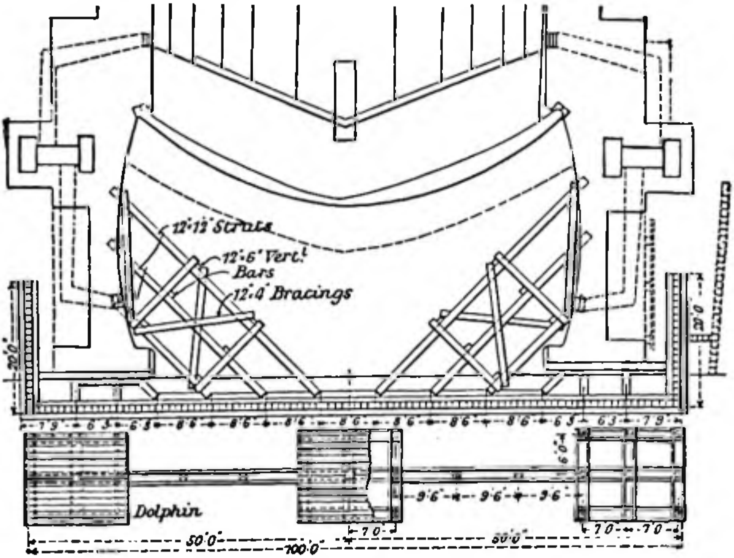


FIG. 19.—Plan of Coffer-Dam, Mercantile Dry Dock Co., River Tyne.

by the onslaught of the sea.[1] Reparation of damaged structures of this class almost invariably necessitates a departure from the original design, and therefore it is difficult to give specific data of any value in the preparation of estimates, as each case merits special consideration. If the damage is at all extensive, the reparation contract is, in many respects, comparable to a contract involving new works. Considerable information as to the cost of construction of mass concrete and masonry sea-walls is given in Professor E. R. Matthews' valuable text-book, which contains useful data as

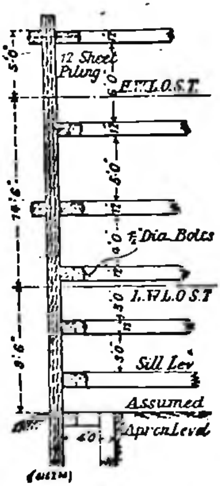


FIG. 20.—Strutting of Coffer-Dam.

[1] *Vide* "The Maintenance of Foreshores," Messrs Crosby Lockwood & Son, pp. 43 and 44. "Sea-Walls."

to costs, profiles, and general design of sea-walls of this character.[1]

The Marine Drive, Scarborough, recently suffered extensive damage during construction, and the continual depredation of the sea at Bexhill at one time became almost historic. The reconstruction and maintenance of such sea-walls and parades are, however, matters which in general are dealt with by the permanent officials of those municipal or urban bodies responsible, and it is only when the wreckage to the sea-front has been considerable that an expert report is called for, and the report then generally includes a recommendation to undertake new protective works combined with a system of groyning. The only useful information in assessment cases of this character is, therefore, the tabulation of actual costs per lineal yard involved in the construction of sea-walls and groynes. A few examples of interest in this respect are given in Table II. It is not, of course, possible within the scope of this chapter to give detailed sections or quantities of works of this character; but, as in the case of sea-walls, the conditions as to exposure, nature of foundations, rise and fall of tide, etc., are so variable that unit prices are useless without a knowledge of the type and leading dimensions. Reference is, therefore, given in the table to the source from which the information was obtained, where full particulars may be found.

The sea and river defences of low-lying lands in general outside the control of municipal authorities represent an enormous mileage in this country — a fact not generally appreciated. Such river and sea defences are peculiarly liable to damage, as, for example, the 90 miles of coast under the control of the Maldon-Wivenhoe and Clacton Drainage Board, Essex. The breaching of such embankments by high tides or wave impact is a fruitful source of destruction. Figs. 21 and 22 show a foreshore section and plan of a breach in such an embankment on the River Deben, in Suffolk.

The author, who was asked to advise in this matter, prepared plans for the closing of the breach, and, at the request

[1] *Vide* "Coast Erosion and Protection," by Ernest R. Matthews, Assoc. M.Inst.C.E. Messrs Charles Griffin & Co. Chapters IV. to VII. inclusive.

of the landowner, obtained tenders for executing the works. The procedure it was proposed to adopt consisted of the driving of a timber coffer-dam of horse-shoe shape in plan on the foreshore outside the breach, and of reconstructing the embankment on the new alignment. In breaches of this

TABLE II.—SEA-WALLS AND GROYNES: COSTS OF CONSTRUCTION

Site.	Structure.	Pre-War Cost of Construction per Lineal Yard. £ s. d.	Reference.
Sewerby -	Timber sea-wall - -	9 16 8	Proceedings of the Institution of Civil Engineers, vol. clix., p. 65.
Scarborough	Sea-wall (Marine Drive). Concrete blocks and mass concrete	73 3 0	Matthews on "Coast Erosion and Protection," p. 58.
Lowestoft -	Sea-wall. Mass concrete on steel piles	5 11 6	Proceedings of the Institution of Civil Engineers, vol. clxxxv., p. 106.
Bridlington -	Alexandra sea-wall. Mass concrete	16 12 9	Proceedings of the Institution of Civil Engineers, vol. clix., p. 65.
Southwold -	Timber groynes - -	4 4 0	Carey on "The Protection of Sea Shores from Erosion," p. 23.
Hornsea -	Sea-wall. Reinforced concrete	39 0 0	Proceedings of the Institution of Civil Engineers, vol. cxxxv., p. 151.
Withernsea -	Timber groynes - -	3 7 6	Proceedings of the Institution of Civil Engineers, vol. clix., p. 73.
Lowestoft -	Timber main groynes (North Denes)	22 1 3	Proceedings of the Institution of Civil Engineers, vol. clxxxv., p. 106.
Bridlington -	Low timber groynes -	2 14 0	Matthews on "Coast Erosion and Protection," p. 115.
Blue[1] Anchor	Timber stockades and groynes	25 10 0	Chapter VI.

[1] Post-War Costs, 1921.

character one invariably finds that a deep *pot-hole* has been cut in the centre of the breach by the action of the tide, which renders new works on the old alignment prohibitive, owing to the expensive nature of the design necessarily involved by such a scheme of works. The average width of the breach in question was 65 ft., and the lowest tender £650.

This breach is only one example of many which exist on the rivers and estuaries of the East Coast, notably at Fingrinhoe Marshes (see Fig. 81), and the cost of reparation of such

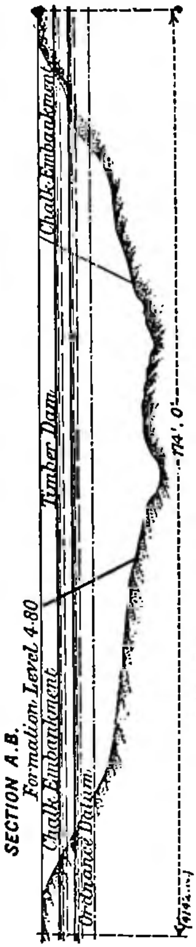


FIG. 21.—Section of Breach, River Deben, Suffolk.

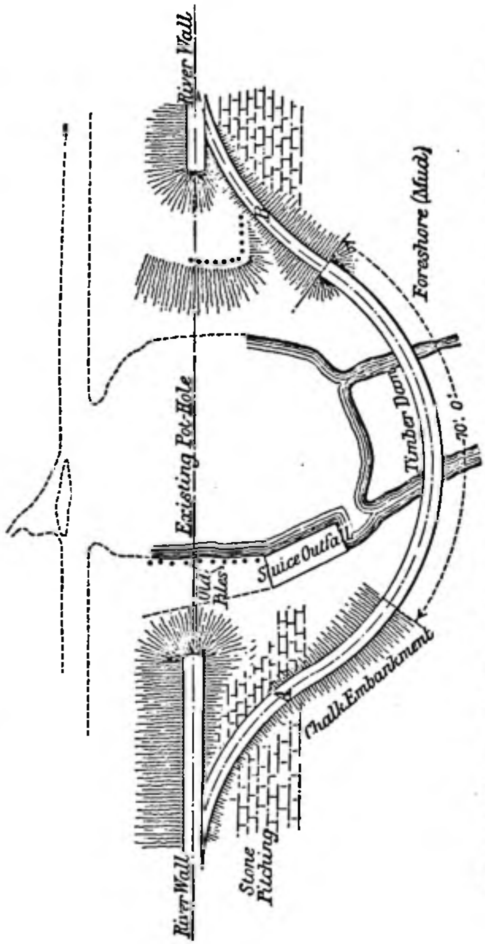


FIG. 22.—Plan of Projected Repair of Breach of Embankment, River Deben, Suffolk.

breaches would therefore appear to be about £10 per lineal foot, the site in question presenting the average difficulties of reparation which exist in breaches of this class. This figure, however, is a speculative one, unless the greatest precautions are taken in the execution of the work.

Fig. 23 is a typical illustration of the profile of an Essex embankment. The maintenance of such embankments is of great importance in assessment cases. In obtaining evidence for an arbitration in 1914 on behalf of the War Office, the author had occasion to investigate this point. The only information which it appeared possible to obtain would seem

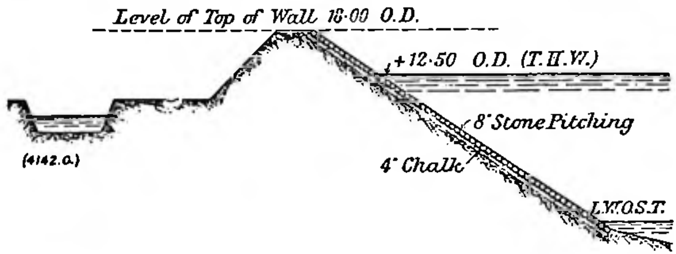


FIG. 23.—Section of an Essex Embankment.

to point to the fact that clay embankments of this character, protected either by stone pitching or a light reinforced concrete revetment, cost before the war about 4.1d. per lineal yard per annum to maintain. The examples given in Table III. are actual cases of maintenance charges taken over short periods.

TABLE III.—MAINTENANCE RATES ON EARTHEN EMBANKMENTS

Site.	Date.	Period.	Cost of Maintenance.	Length of Embankment.	Rate per Yard per Annum.
		Years.	£ s.	Yards.	d.
Embankment, South Level (East Section). Fobbing Level Commissioners	1910-12	3	492 0	11,400	4.34[1]
Anglo-Saxon Petroleum Company. Sea Defences, Thames Haven	1912-13	2	38 0	1,100	4.14
River Embankment, Aldeburgh, River Alde. Hurts Hall Estate	1910-12	3	27 10	593	3.71

[1] Information supplied by Mr A. E. Carey, M.Inst C.E., Engineer to the Fobbing Level Commissioners.

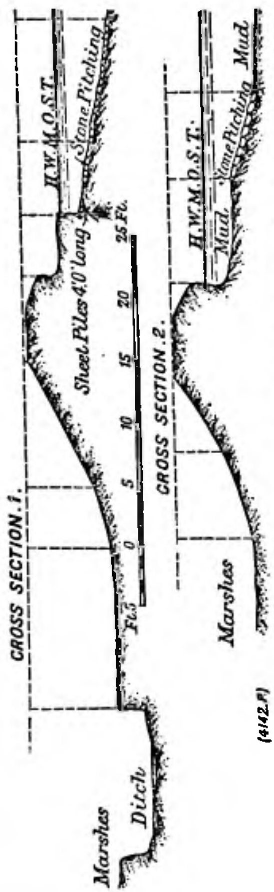
In these figures no abnormal charges for new works were included, and it must be remembered that they only held good in cases where embankments were in normal repair.

It is frequently necessary to prepare estimates as to the cost of putting such defences into an efficient state. Table IV. is an actual estimate supplied by the author in respect of some property acquired by the Government on the River Alde.

TABLE IV.—SCHEDULES OF REPAIRS TO RIVER EMBANKMENT, RIVER ALDE, SUFFOLK

Section.	Length in Yards.	Work Necessary.	Length of Work (Yards).	Rate per Lineal Yard. s. d.	Cost. £ s. d.
I.	490	*Nil*	...	...	*Nil*
II.	54	Pick up and re-pitch 270 sq. yds. of chalk pitching	54	5 0	13 10 0
III.	237	Make up face with new earthwork. 1⅔ cub. yds. per yard run. Re-pitch 3 sq. yds. per yard run. New pitching 2 sq. yds. per yard run	237	13 6	159 19 6
IV.	393	As per Section III. - -	73	13 6	49 5 6
IV.	...	(*a*) Pitching in rough condition. Allow for re-pitching and making up toe	...	...	30 0 0
IV.	...	(*b*) Re-set coping stones and point up brickwork of sluice	...	...	5 0 0
V.	727	Make up toe. 1 cub. yd. of earthwork per yard run, 2 sq. yds. new pitching per yard run	100	9 6	47 10 0
VI.	217	Patch up earthwork - -	...	...	5 0 0
VII.	610	Natural boundary of shingle ridges	...	...	*Nil*
VIII.	210	Re-form bank with additional earthwork	210	...	10 0 0
IX.	400	Re-pitch 1 sq. yd. per yard forward. New pitching 1 sq. yd. per yard forward	200	5 0	50 0 0
X.	503	1⅔ yds. per cub. yd. new earthwork per yard forward; 3 sq. yds. new pitching per yard forward	171	14 6	120 19 6
X.	...	Patch up earthwork on face where required	...	...	10 0 0
In addition to above, topping and trimming 3,231 lineal yds. of embankment				0 6	80 15 6
					582 0 0
Add for contingencies on maritime work, 15 per cent.				...	87 6 0
					669 6 0
Supervision, 5 per cent. - - - - - - -				...	33 9 4
Total estimated cost of work - - -				...	702 15 4

The schedule is of interest as giving unit rates for repair work of this character. Figs. 24 and 25 give two average cross sections of the walls referred to in the estimate, which sections show seriously eroded profiles, and are typical of the condition of many existing clay embankments in Kent, Essex, Suffolk, and Nórfolk.



FIGS. 24 and 25.—Cross Sections of Embankment on River Alde.

The condition of protective works of this nature has a most important bearing upon the values of the properties they defend, and, apart from the question of damage to crops, and depreciation in the agricultural value of land caused by flooding, is the liability which most walls of this character have to be breached. There is, in the author's opinion, no question but that some specific sum in addition to the annual cost of maintenance should be set aside each year to meet such extraordinary expenditure as may accrue in any one year, due to the recurrence of a breach in the embankments. This can only be arrived at by ascertaining the historical record of neighbouring embankments. In the case quoted above, breaches had occurred in adjacent walls in 1877 and 1897, and a fair assessment in this case appeared to be the sum of £20 per annum,[1] over a total length of 3,231 yds. of river embankment. Embankments of this

[1] *Vide* Arbitration. Lord Rendlesham and Secretary of State for War, 1914. Question 1897.

class, more especially where they abut on the coast or an exposed estuary, are frequently subject to scour at the toe.

Fig. 26 is a photograph illustrating such a type of defence.

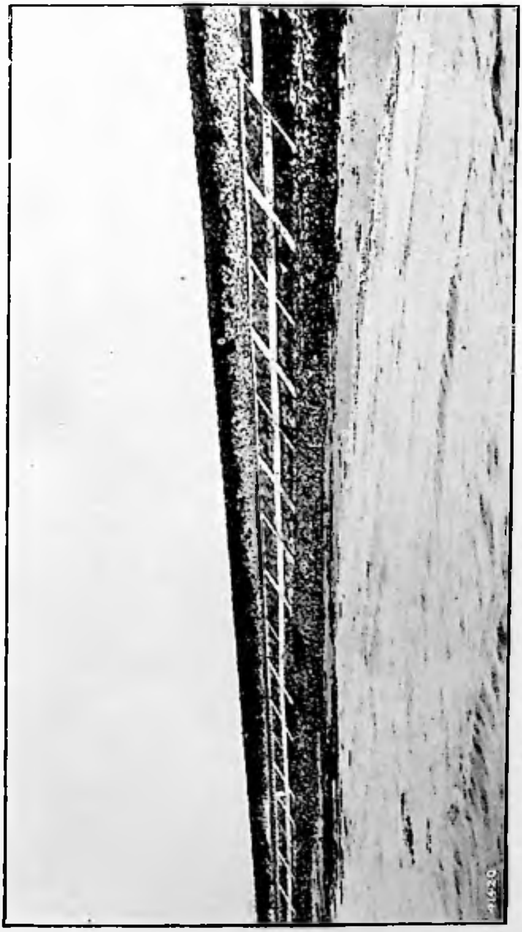


Fig. 26.—Reinforced Revetment on the De Muralt System.

In this case the protected saltings in front of the bank had become eroded by scour, and it was necessary to protect the face in order to maintain the stability of the bank. To this end a reinforced revetment on the De Muralt system was

employed. The work was carried out in January 1913. The revetment has a slope of $1\frac{3}{4}$ to 1, and varies from 4 to 5 yds. in depth. It is 394 yds. long, and was con-

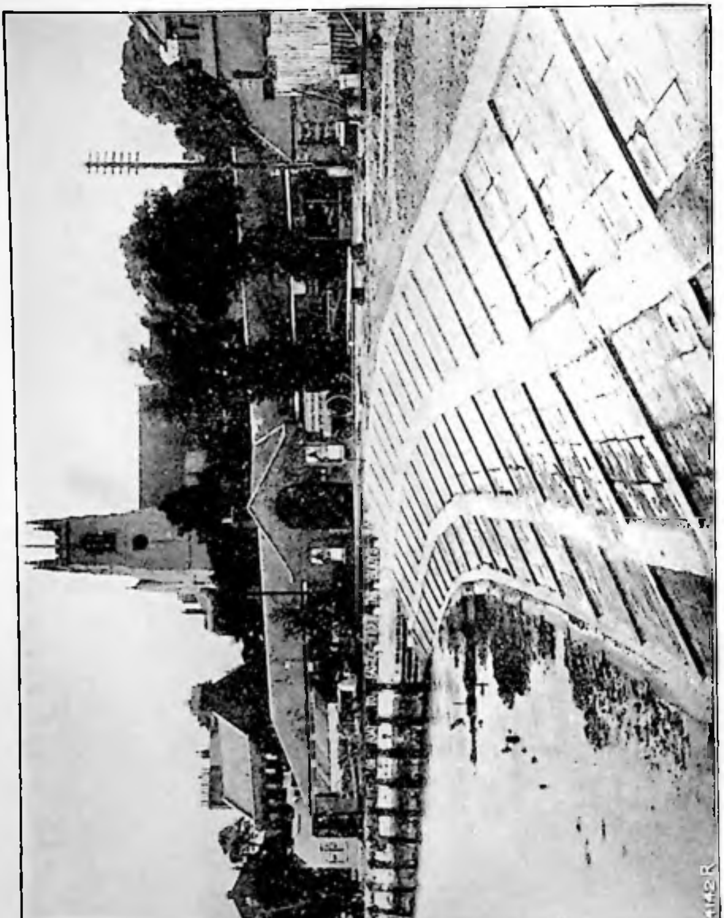


FIG. 27.—Reinforced Revetment on the De Muralt System on the Medway.

structed for a total cost of £1,767. Whereas the initial cost is low, the maintenance charges of such work in exposed positions are likely to be high. In the case in question a sum of approximately £300 has been spent to date (January 1915) in maintenance, which approximates to the sum of

7s. 6d. per lineal yard per annum. It is interesting to note that a subsequent estimate for permanent works on this site exceeded £12,000; and, in spite of the somewhat heavy

FIG. 28.—Eroded Toe of Wall at Frinton.

maintenance charges, such a light revetment would therefore seem to be of use where regular inspection of the work is possible, but the cost of maintenance, referred to above, must always be borne in mind when dealing with assessment questions.

Table V.—Schedule of Assessment Cases. (1906-15)

Nature of Structure and Site.	Date.	Colliding Vessel.	Cause of Accident.	Damage to or State of Works.	Advisory Engineers.[1]	Cost of Reparation.
Reinforced concrete jetty, Dagenham	2 A.M., 25th Feb. 1911	Primarily S.S. *Florence*, 200 tons register. Secondarily barge *Fred*, 1057	Vessel going down river tried to avoid H.M.S. *Thunderer* lying at jetty and ran into barge lying alongside jetty approach	Two reinforced concrete piles fractured at junction with superstructure	A. E. Carey, M.Inst.C.E.[2]	£150 (estimated).
Pitch-pine timber jetty, Tilbury	5th Jan. 1914	S.S. *Ryhope*, 1,700 tons burthen	Collided whilst manœuvring for berth	One main pile fractured and damage to short length of capping-piece	The author	£80 (contract).
Falseworks (timber and mild steel) and plant, Admiralty Harbour Works, Dover	Oct. 1906	S.S. *Olaus Ollsen* (1,200 tons register)	Vessel collided with gantry when making entrance to harbour in fog	Demolition of three bays of gantry and wreckage of considerable plant	A. E. Carey, M.Inst.C.E.[2]	£27,000 (claim, approximate).
Timber pier head, Clacton	Feb. 1914	...	...	Natural decay of Memel fir piles. Twelve replaced with six cast-iron columns and mild-steel girders	The author.	£650.
Continuous iron lattice girder road bridge, Wandsworth Bridge. Carried over river on piers each consisting of two wrought-iron columns filled with concrete. Built 1873	19th June 1912	S.S. *Wandle*, 889 tons (gross)	Vessel navigating at slow speed in foggy weather in tideway when approaching berth above bridge	One main column fractured and displaced 5 ft. 6 in. at head and severance of cross-girder connections	A. E. Carey, M.Inst.C.E.[2] H. Wilson Holman, M.I.N.A. The author	£4,247 (claim).
Jarrah and pitch-pine timber jetty, River Thames	1911	7,816 tons (gross)	Doubtful. Vessel out of course passed through jetty approach on top of tide at night	Twelve main piles fractured. Moorings and buoys cut adrift. Approach to jetty completely severed. Five bays of 15 ft. span demolished; four bays of 15 ft. span strained.	...	£1,420 (contract). Other claims for consequential damage.
River defences, King's Marshes, Orfordness	...	...	...	Neglected condition of 3,231 lineal yds. of clay embankments protecting site of aviation ground from flood	The author	£700 (and £1,300 capitalisation as to future maintenance).

Iron swing bridge, Poole	6th March 1908	S.S. *White Heather*, 333 tons (gross)	...	Damage to piling supporting swinging span and protective dolphins and fendering. Severance of gas main	H. Wilson Holman, M.I.N.A.	£2,037 (contract and disbursements).
Reinforced concrete deep water jetty, Dagenham Dock	April 1907	S.S. *Goorkha*, 6,300 tons (gross)	Foggy weather; vessel reported to have been deliberately put ashore to avoid collision with small steamer	Partial severance of jetty approach. Demolition of one bay and fracture of most struts and bracings in two adjacent bays	A. E. Carey, M.Inst.C.E.[2]	£1,600 (estimated).
Pitch-pine timber jetty, Purfleet	4.30 A.M., 5th March 1908	S.S. *Harrovian*, 3,307 tons (gross)	Failure of steering gear while vessel was proceeding down river on ebb tide	Three 7 ft. bays and upstream nose severely stressed and set over 2 ft. at deck level	H. Wilson Holman, M.I.N.A. A. E. Carey, M.Inst.C.E.[2]	£1,100 (settlement).
Timber face of tidal wharf, Surrey Commercial Docks	March 1907	S.S. *Narva*; length, 275 ft.; beam, 55 ft.	Vessel aground at low water; "mud sucked" on rising tide; snapped cables and surged stem-on against wharf face	Wharf face driven inwards by impact, and camp-sheeting fractured	A. E. Carey, M.Inst.C.E.[2]	£50 (estimated).
Steel caisson dock gates; dry dock, River Tyne	2.30 P.M., 23rd Dec. 1908	S.S. *Patris*, 2,500 tons (displacement)	Accidental severance of cable and fracture of drag-box shackle during launch of vessel from opposite bay of river	Complete demolition of one leaf of dock gate, and slight straining of other leaf; damage to dock sill and to steamer in dock	J. Mitchell Moncrieff, M.Inst.C.E.[2] A. E. Carey, M.Inst.C.E.[2]	£7,500 (settlement as to works), and claim for consequential damages.
Reinforced concrete jetty, River Thames	1912	4,241 tons (gross)	Foggy weather; vessel proceeding up-stream off her course and to the north of the fairway	One reinforced concrete 5 ft. diameter column, fractured about 10 ft. above low water; penetration of decking and superstructure for 17 ft.; several reinforced concrete columns cracked near bed of river by impact	...	£2,700 (contract).
Concrete and masonry dock wall, Queen Alexandra Dock, Cardiff	8th Dec. 1907	S.S. *Llansannor*, 2,307 tons (register)	Failure of screw mooring when subjected to shock by ship's cables parting, due to wind pressure on freeboard of ship lying to buoys in open dock	Slight damage to dock wall and steamer *Corn Exchange*	A. E. Carey, M.Inst.C.E.[2]	About £800 (claim).
Clay embankment, River Deben	April 1914	...	...	Breach in embankment: average width, 65 ft.	The author	£650 (tender).
Reinforced concrete apron, Frinton	1913	...	...	Toe of stone-protected earthen embankment damaged by sea for distance of 200 yds.	A. E. Carey, M.Inst.C.E.[2] The author	£450 (approximate).

[1] These references are restricted to those engineers acting for one or other of the principals concerned who have supplied information for the present volume.
[2] To whom the author was chief assistant.

Fig. 27 illustrates a similar reinforced concrete revetment on the banks of the Medway at Chatham. This revetment protects a length of 90 yds. of bank between two wharves, and was constructed in 1913 for an inclusive cost of £7. 15s. 5d. per lineal yard.

Fig. 28 is a photograph showing the condition at the bottom of a stone pitched wall at Frinton, on the East Coast, where the undermining effect caused by the sea is apparent. To protect this embankment a reinforced concrete apron was constructed, a cross section of which is shown in Fig. 29.

The work was completed early in 1913. It extends

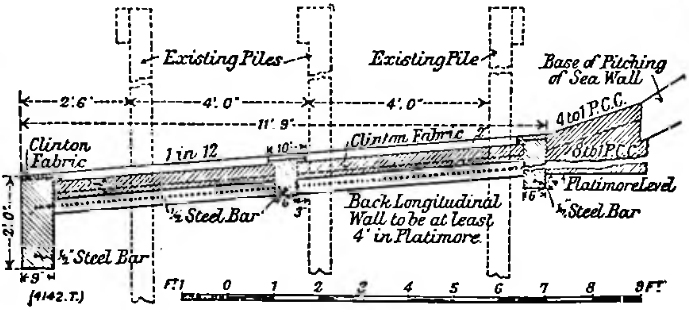


FIG. 29.—Reinforced Revetment at Frinton.

for a distance of 200 yds. and the construction has cost approximately £2. 5s. per lineal yard.

The author has prepared a general summary of selected assessment cases dealt with between the years 1906 and 1915, which may be of use as a general reference; this summary is given in Table V. In cases where damage was caused by a colliding vessel, the tonnage is quoted in column 3 from data which was obtained at the time of the accident. In this respect, in dealing with such assessment cases, it is vital to avoid any confusion between "tons burthen," "tons register," "tons displacement," and "dead weight." The two former are only of value as giving some indication of the size of the ship; the latter, together with the dead weight of cargo carried, is, of course, the figure upon which all impact calculations must be based, measurement tonnage being in most cases extremely misleading.

CHAPTER III

THE MAINTENANCE OF TIDAL BERTHS[1]

Dredging and Maintaining Berths—The Fendering of Different Types of Structures—Considerations in respect of Fendering Cylindrical Structures—The Disintegration of Reinforced Concrete Structures—Marine Structures—The Decay of Timber—The Necessity of Periodical Surveys.

THE maintenance of tidal berths is a subject of considerable importance. There are several factors which are frequently overlooked and form the causes of damage either to the berth itself or to shipping. The subject matter of this chapter really forms the border line between the functions of the surveyor expert in civil engineering matters and the marine surveyor.

In the case of any berth, careful consideration must be given as to whether or not a jetty, quay, or wharf in a tideway is being constantly damaged, involving considerable expenditure. The cause may often be found in faulty mooring arrangements. Experience has shown that civil engineers in general are not sufficiently acquainted with the technicalities of mooring ships. Whereas it is probably no part of their duty to acquaint themselves with the technicalities of Lloyd's requirements and those of other insurance bureaux, it is desirable that the general aspect of the question should be studied, particularly as to the varying angles of altitude and depression on the different ropes which occur as the ship rises and falls with the tide. This is the great disadvantage of all tidal berths and does not, of course, occur to the same extent in a dock floatage.

[1] Reprinted from an article in the *Engineer*, 27th January 1922.

Dredging and Maintaining Berths

At most tidal quays the ships berthing are either ashore at low water or only *partially* water-borne. Regular inspection, therefore, of the river bed forming the berth is a matter of great importance, and in nearly all cases the berth must have definite attention either by the dumping of small block chalk and levelling or by dredging, dragging, or grabbing. It is remarkable in some riverside trades how rapidly obstructions build themselves up on the river bed. In the case of one large corporation owning several properties on the River Thames, there was reason to suspect that a ship of nearly 6,000 tons dead weight had strained herself on a berth where there were presumed to be about 10 ft. of water at low tide. Examination of this berth showed that the cranage facilities did not permit of the ship remaining in one position during the whole period of discharge; further, the berth was of such importance that between 300 and 400 ships visited it per annum, or roughly one ship every other tide. The necessity of putting this berth in sound condition, therefore, was very urgent. Emergency soundings were taken at once, and certain inequalities in the bed of the berth, as also certain isolated patches of hard material, were noted. The surveyor's recommendations were ultimately carried out, and these consisted of dredging the berth to a uniform depth, involving the removal of over 6,000 cub. yds. of material. A detailed survey was afterwards made which indicated that one of the hard "patches" had not been successfully removed by the dredger. A diver's inspection was made and it was found that this "ridge" or "reef" ran right across the centre of the berth. Arrangements were, therefore, made to break up the hard material, which consisted of ballast "cemented" together so as to have almost the consistency of concrete. The reef was ultimately removed by an improvised rock breaker and by grabbing up the broken material. The reef was approximately 40 ft. wide and 50 ft. long, and just how it came to be formed was a matter of conjecture.

If local patches of hard material have to be removed it is of great importance to ensure that the excavated level is

below the general level of the berth, and, if there is no silt or mud travel, the artificial depression so formed must be filled up with clay or other suitable material. A common mistake, and frequently an expensive one, is to dump chalk or soft limestone in large fragments in a haphazard manner and with no proper attempt at levelling the berth afterwards. The motor schooner *Clarita* was damaged in 1920 at Odams' Wharf on the Thames-side; in this case the damage necessitated the ship being run ashore shortly after starting, and it was not possible for the wharfingers to put forward a really sound defence.

Although there are exceptions, as a general rule a wharfinger has to provide a fair berth. In the case of a damaged ship the legal aspect is somewhat complex and uncertain. On a vessel being found to have sustained bottom damage, it is frequently assumed by the owners that this has been caused by the last "dry" berth occupied by the ship. This was the case made by the owners of the S.S. *Irkutsk* against Hayes' Wharf, London Bridge, in 1909. A complete survey of the berthage was made, which revealed nothing of a nature to account for the extensive damage to the ship. The ship was new, double-bottomed with closely spaced diaphragm plates, being specially designed for taking the ground. The outer skin of the ship was badly set up and numerous diaphragm plates buckled. In this case the wharfingers stoutly denied the suggestion of a foul berth, and it was ultimately ascertained that the ship had suffered damage elsewhere. This is the class of case which the maritime engineer is often required to deal with, and the problems presented must be approached with care.

In the case of alleged damage to a ship through taking the ground there are two main points to bear in mind:—

1. Was the ship of suitable design to occupy a dry or partially water-borne berth?

2. What was her exact location on the berth, and was there any corresponding hard obstruction on the bed which could in any way have caused the alleged damage?

As regards (1) this should be at once cleared up by a survey of the ship, and it cannot be too clearly emphasised

that this is a matter for a naval architect or marine surveyor, and does not fall within the province of the ordinary surveyor or civil engineer. As regards (2) it is seldom the *exact* position can be obtained. The author is only aware of one firm on the River Thames, *i.e.*, the South Metropolitan Gas Company, which actually charts the position of each ship berthing—a most desirable thing to do. If the berth is entirely dry at low water a careful inspection can be made, and it is essential to go over the berth with an iron probe. If the berth is not dry, soundings must be taken and a diver should, in addition, be sent down. The position of any sounding or level taken must always be accurately defined with regard to the shore line as, in the event of litigation, the ability to vouch for accuracy of "position" is equally important to both sides in establishing the "facts." The frequency with which cases of damage to shipping occur through ships taking the ground at tidal berths is in itself the strongest recommendation in favour of periodic surveys.

Fendering

One of the biggest problems of maintenance is presented in the ever-recurring damage to fendering, and the whole subject really merits much more consideration than it has been hitherto accorded by civil engineers. The fendering of tidal berths is divided into three classes, viz.:—

1. The fendering on the face of a solid masonry, concrete, brick, or close-timbered quay face.
2. The fendering of an open piled structure.
3. The fendering of jetties constructed on the "cylinder" system (*i.e.*, large diameter piers and large spans).

The first class of fendering calls for little remark, and usually consists of vertical timbering at close intervals. The fenders are sometimes faced with half round steel bands similar to a ship's rubbing pieces, and horizontal distance pieces are sometimes provided. As replacement is frequently necessary, bolting through into the quay face should be avoided. When such fenders are carried away by ships or barges the bolts are nearly always bent or otherwise strained, and putting in new bolts to a solid quay face to which there is

no access behind is a waste of money and time. Such fenders should preferably be strapped, the strap being designed so as to be considerably weaker than its attachments to the quay face, thus, in the case of damage, ensuring only the failure of the straps, which can quickly be renewed.

The second class of fendering is generally most unsatisfactory, and one hesitates to estimate the very heavy annual expense which occurs through fenders being "carried away or displaced." These costs appear to be fairly equally divided in the long run between the wharfingers and the shipowners, but they are a continual source of annoyance to both.

The design of fendering for open piled structures on the River Thames—see Fig. 30—is more or less standardised, but it does not appear to be efficient. The fendering is usually in English or American elm and of 12 in. by 6 in. section, sometimes heavier. The spacing of the fenders is governed by that of the main piles, varying from 8 ft. to 15 ft. The fenders are generally bolted through the main piles with bolts about 5 ft. apart, though in the case of reinforced concrete piles they are sometimes strapped on with steel straps. Opinion seems to be divided as to whether the fenders should be bolted straight or strapped on to the face of the main piles, or whether they should have small distance pieces about 6 in. thick at each level where the bolts or straps occur, leaving the fenders elsewhere clear of the main pile. The writer is in favour of the fender either coming right against the face of the main pile or having continuous packing behind it. Failure occurs nearly always in the same way; a ship, berthing, catches the projecting fendering with the proud ends of her plating or other projections, and the fenders snap off at the weakest points, *i.e.*, at the bolt holes or where recessed to receive the straps. In some cases additional horizontal fendering A, A, shown by the dotted lines in Fig. 30, has been resorted to. This certainly gets over one objection, but, nevertheless, introduces another owing to the liability of projections from the ship catching under or resting on this horizontal fendering as the tide respectively rises or falls. In general, however, the addition

of horizontal fendering is to be favoured, and it possesses some value in transferring lateral stresses to the vertical fendering. In nearly all classes of fendering English and

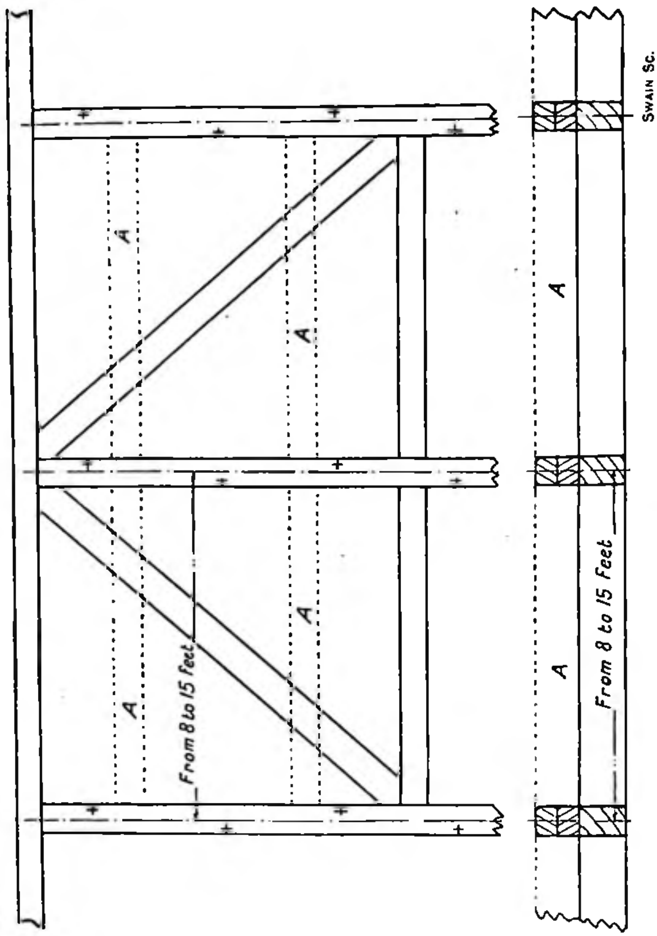


FIG. 30.—Fendering for Open Piled Structures (River Thames).

American elm is usually employed in this country for fendering open piled structures in the sea or rivers. Where the landing stage or jetty is constructed in steel a neat arrangement is to insert timber fendering between the flanges of a channel iron, which, in turn, may form a principal

member of the stage itself or is readily attachable by bolts to the main structure. This method was adopted some time ago on the landing stage of the West Pier, Brighton, and appears to have given satisfaction.

Fendering Cylindrical Structures

The fendering of cylindrical structures presents the greatest problem of all. The writer has experience of three existing cases:—(*a*) No. 1 jetty head, Thames Haven (London and Thames Haven Oil Wharves, Limited). (*b*) Coal and coke berths, East Greenwich (South Metropolitan Gas Company, Limited). (*c*) Jetty of the Imperial Paper Mills, Gravesend.

The conditions at the above jetties were in 1922 as follows:—

Thames Haven.—Reinforced concrete cylinders, 20 ft. centres, protected by timber piles and staging on the face. Ships do not, therefore, come alongside the cylinders. The cylinders are 5 ft. diameter, changing to oval section above low-water level.

East Greenwich.—Steel cylinders, 53 ft. centres, 6 ft. diameter. Ships are boomed off by built-up timber booms floating against the face. Ships, therefore, do not touch these columns.

Gravesend.—Reinforced concrete cylinders, 25 ft. centres, 5 ft. diameter; 12 in. by 12 in. horizontal timbers are attached to the cylinders at three levels by brackets. In front of these 12 in. by 12 in. timber piles are driven and bolted to the horizontals with horizontal blocking pieces in the intervals. Elm fenders (12 in. by 6 in.) are bolted on to the driven piles. This method is really equivalent to putting the face of a timber jetty outside a reinforced concrete structure. This fendering is illustrated in Fig. 31. It is reported on by the resident engineer (Mr A. Wheeler) as effective but expensive.

Enough has probably been said already to indicate that existing methods of fendering tidal berths leave much to be desired. Experiments with new methods are likely to be expensive, and the civil engineer is naturally shy of recom-

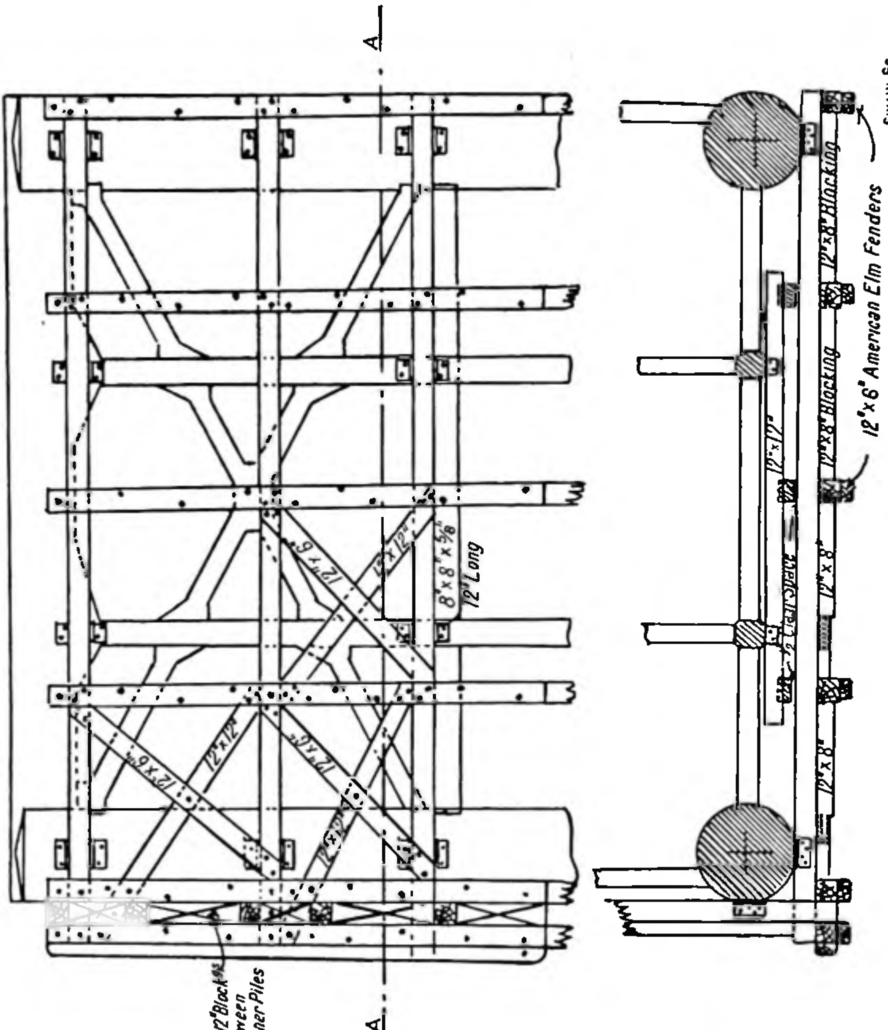


FIG. 31.—Fendering for Reinforced Concrete Jetty, Gravesend.

mending this. The author believes that the best solution will be found to lie in vertical fenders of circular section capable of rotating. They should have a longer life and be much less liable to be carried away.

Concrete Constructions

Apart from the necessity of keeping the river bottom in good condition and maintaining the fendering, the actual maintenance of the jetty, pier, or wharf is often neglected, and for the sake of saving a small annual expenditure a structure in the river or sea has often to be almost rebuilt after a few years' neglect. Of late years there has been a growing tendency to construct marine works in reinforced concrete in preference to timber, steel, or even masonry, on the ground of its great durability. The behaviour, however, of reinforced concrete on land is materially different to its behaviour as part of a marine structure, and Mr L. H. Savile, M.Inst.C.E., gave the experience of the Admiralty in a very thoughtful communication made to the Institution of Civil Engineers at the Engineering Conference, 1921 (Sec. II., Harbours, Docks, Rivers, and Canals). He stated that the British Admiralty had experienced a considerable amount of deterioration in reinforced concrete piers, wharves, etc. One of the most serious facts established by Mr Savile was that reinforced concrete deck beams well above high-water level showed signs of failure and, from photographs produced at the conference, there seems little doubt that reinforced concrete structures require the same periodic inspections as do those of either timber or steel. Colonel Mitchell Moncrieff, M.Inst.C.E., as the result of inspecting a number of reinforced concrete marine works, has obtained much valuable information as to the poor condition in which these structures are frequently to be found. As regards the normal "decay" of reinforced concrete work, Mr Savile offers several well reasoned suggestions as to the causes of disintegration of reinforced concrete in marine works. It is safe to say that his conclusions as to the *porosity* of concrete are probably correct, and, in the case of new construction, rich mixtures of concrete should be adhered to. It is a safe general practice not to permit any weaker mixture than 4 to 1 by volume, *i.e.*, 4 parts of mixed aggregate and sand to 1 part of Portland cement concrete, although, according to the nature and size of the aggregate and sand particles, richer mixtures may frequently be called for. The author is

inclined to think that in the case of marine work in reinforced concrete it is desirable to pickle and clean the steel reinforcement before its introduction to the structure.

More importance should be attached to the design of the decking of any jetty, wharf, etc., than is usually the case. In reinforced concrete structures the top skin of the deck should be as nearly as possible an impervious surface consisting of a very dense granolithic mixture or of asphalt. If the deck is practically air- and water-tight there will be less chance of "decay" occurring in the deck beams. If an artificial asphalt surfacing is employed, it is important that it should be applied to a rigid specification. In the case of repairing "decayed" or damaged members in reinforced concrete, it should be emphasised that mere surface patching with sand-cement grout is useless, and that the whole concrete, wherever cracking or flaking has occurred, must be cut away for some distance back on either side, and the whole member recast.

Decay of Timber

In the case of timber structures in this country, decay usually commences in the deck bearers by reason of their holding moisture which runs off from the deck. In the cases of tight-jointed decking this decay in the deck bearers is delayed, but the deck itself will suffer. Considerable attention should be paid to the condition of the deck bearers, and sections of the deck should be periodically lifted and the bearers examined. The writer has found that where the deck bearers are only slightly decayed on their upper surfaces, a coating of asphalt will prolong life. In all new work it is highly important to design timber structures so that the deck bearers can be easily renewed without having to dismantle a great deal of the rest of the superstructure. As regards the preservation of timber, there are many trade articles advertised, but experience has shown that careful creosoting with the best creosote oil is so far the best known preservative for such timbers as pitch pine, Oregon, etc. Harder timbers, such as jarrah, karri, greenheart, elm, oak, etc., will not absorb creosote to any extent, and their preservation depends on periodical tarring or painting. Tar or

pitch should always be applied hot, and all timbering standing in the sea or river should receive two coats of tar every three years. There are many methods of creosoting :—

1. By allowing the timber to soak in creosote oil.
2. By carefully drying the timber beforehand and creosoting under pressure.
3. By putting the timber in a vacuum before and after creosoting under pressure.

A variation of (3) is adopted by the General Post Office in all their specifications. In general it will be found sufficient to creosote timber in tanks under pressures varying from 150 lbs. to 200 lbs. per sq. in. It is equally important to creosote any new piles or members which have to be employed in repair work. There is, too, frequently a tendency to employ new timber in repair work, and to overlook creosoting because the quantity of timber is so small. If all timber is not thoroughly impregnated with creosote or kept well tarred, rapid decay will occur.

The life of steel between wind and water is at least as long as that of timber, probably longer. A great deal of variation occurs in the rate at which steel rusts. In cases in which the structure is not properly maintained by painting or tarring, the variation in the rate of rusting seems to depend on the salinity of the water and the carbon content of the steel. In general, the lower the carbon content the less the rate of rusting, and this is well evidenced by the fact that the tail shafts of ships are usually constructed in wrought iron for the sake of durability. It is essential that steel structures should also be regularly examined, and in general it will be found that trouble occurs first at the bolt holes, the bolt rusting and becoming smaller in diameter and the bolt hole larger. The joint eventually becomes loose, causing the structure to "work." If this is closely watched and taken in time, considerable sums of money can be saved, because otherwise the whole member, which is very often in quite good condition, being only eroded badly at the bolt holes, has to be replaced. In new construction the bolts, nuts, washers, etc., should be clean before assembly, and both the bolt hole and the bolt, etc., thoroughly tarred before use.

In the case of fractured or damaged piles, in order to avoid the cost of dismantling a large portion of the structure, it is often practicable to drive a new pile alongside the old one and bolt or strap the new to the old pile. This cannot, of course, be done in the case of reinforced concrete structures, although it may sometimes be done in the case of steel structures. In the case of wharf walls with a solid fill behind them, if there are any anchor ties, the ground round them should be occasionally opened up and these ties examined, as many walls fail by bulging outwards as the ties decay.

It is hoped that one or two useful points may have been brought out in this chapter, but the principal point for the wharfinger to bear in mind is not to go on using a berthage until a serious accident happens, either to shipping or to the structure itself, but to have both the bed of the berth and the structure periodically surveyed and examined by a responsible civil engineer. It is, of course, in the nature of things that the engineer will almost certainly report that certain work is necessary as a result of each inspection, but whether or not any repair work is carried out then rests with the wharfinger. If he does not carry out the work, he at any rate knows what the condition of the structure is, and can judge as to its safety and the risks of carrying on without effecting repairs.

The author has known cases in which a jetty or quay has been standing without attention for a number of years, and then some small repairs have at last been decided on. Directly the work of dismantling has been started, reconstruction on a far more extensive scale is usually found to be necessary. Needless to say, they have all been cases in which the absence of regular inspection has been either through neglect or an "economy."

CHAPTER IV

PILE-DRIVING[1]

Difficulties of Recording "Set"—The Driving of Test Piles—Formulæ for Determining Safe Loads—Experiments with Inertia Gauges for Recording "Set."

THE driving of piles in many cases forms one of the principal features of construction in marine works. In order to obtain the necessary data as regards foundations in the sea or river bed it is usual, in cases where there is any doubt, to have borings taken or test piles driven, or to observe both precautions. The writer places greater reliance on the results of test piles than on the results of borings in so far as actual ability of the ground to carry loads is concerned.

If borings are decided on, it is usually wise to have them carried out by one of the several firms who specialise in this class of work; their plant is largely standardised, and they are better able to deal with the commoner delays which are likely to occur, due to broken boring tools or difficulties in driving or drawing the lining of the bore. As regards cost, this is entirely a matter depending on the nature of the strata to be penetrated, and the conditions to be met with on the site. Test borings of this class must be distinguished from boring Artesian wells which for many reasons is obviously a more elaborate proceeding, and these latter wells are, of course, usually carried to much greater depths.

In driving test piles, very careful observation of the driving is necessary, as, in driving from a "springy" stage or from floating craft, owing to the movement of the stage or the oscillation of the barge or pontoon, it is difficult to measure the actual penetration per blow which it is essential to observe

[1] Reprinted from an article entitled "The Gauging of Penetration in Pile-Driving," *Engineering*, 18th November 1921.

closely when driving test piles from the moment when they begin to "pull up."

In driving permanent piles it is unfortunately necessary for practical reasons to average the penetration over a number of blows. This is common practice, but there are technical objections to it, as if a pile has pulled up hard; this may obviously have occurred on the first one or two of a series of blows, and the balance of the series (before driving is stopped for the purpose of observation) may result in the destruction of the pile head or in certain cases, especially in reinforced concrete piles, even in the destruction of the pile itself.

The data which it is necessary to record in test pile-driving is best shown by a record of actual driving which is given below for a single test pile driven through clay and peat with an ultimate "set" in Thames ballast.

TABLE VI.—RECORD OF TEST PILE (OREGON PINE, 14 in. by 14 in.). GREENHITHE, 17TH JANUARY 1921

Number of Blows.	Penetration.	Number of Blows.	Penetration.	Number of Blows.	Penetration.
	Ft. In.		In.		In.
10	8 5 1/2	10	1 5/8	5	3/4
10	4 9 1/4	10	1 1/2	5	1/2
10	4 0 3/4	10	1 1/2	5	7/16
10	3 4 1/2	10	1 3/8	5	1/2
10	2 6 1/2	10	1 1/4	5	7/16
10	2 9 1/4	5	1/2	5	1/2
10	1 9	5	5/8	5	3/8
10	5 1/4	5	1/2	5	1/2
10	3 7/8	5	11/16	5	1/2
10	3 3/8	5	1	5	7/16
10	3	5	3/4	5	7/16
10	2 1/2	5	3/4	5	3/4
10	2	5	11/16	5	11/16
10	2	5	7/8	5	5/8
10	2	5	3/4	5	6/16
10	2	5	3/4	5	5/16
10	1 3/4	5	3/4	5	6/16
10	1 3/4	5	3/4	1	1/16
10	1 3/4	5	1		

Length over all - - -	53 ft. 6 in.
Level of pile head - -	+ 16.49 O.D.
Level of shoe - - -	− 37.01 O.D.
Penetration - - -	42 ft., 6 in.
Level of ground - - -	+ 5.50 O.D.
Final set (as above) - -	1/16 in.

The question of accurately determining the "set" for safe loads which permanent piles may be expected to carry has occupied the writer's attention for several years, mainly with the object of avoiding the technical objections of averaging the final set over a number of blows and the delay in stopping driving for the purpose of observation. As the result of experience in actual driving he prepared an empirical formula in 1914,[1] and to this future contracts under his firm's jurisdiction have been satisfactorily carried out without failure in any case. This formula is dependent on a factor p, as given below :—

TABLE VII.

Final "Set" of Pile per Blow.	Value of Factor p.	
	Driving Without Dolly.	Driving With Dolly.
Less than $\frac{1}{16}$ in.	0.20	0.15
,, $\frac{3}{32}$ in.	0.15	0.12
,, $\frac{1}{8}$ in.	0.12	0.08
,, $\frac{3}{16}$ in.	0.10	0.05
,, $\frac{1}{4}$ in.	0.09	0.03

The safe load is calculated as follows :—

$$B = \frac{L}{p} \qquad (3)$$

where L = safe load in tons.
p = factor as above.
B = driving force of blow in tons calculated as follows :—

$$B \times X = F \times T \qquad (4)$$

where X = penetration per blow in feet at or towards conclusion of driving.
F = fall of monkey in feet.
T = weight of monkey in tons.

The British Steel Piling Company, who have a wide experience in pile-driving, and no doubt working on similar lines, published in their handbook of 1921 a formula having a similar object.

[1] See "Maintenance of Foreshores," published by Messrs Crosby Lockwood & Son, p. 80.

Their formula is as follows:—

$$L = \frac{2wh}{S}\left(1 \times \frac{W}{w}\right) \qquad (5)$$

where L = safe load in pounds.
w = weight of monkey in pounds.
h = height of fall in feet.
S = penetration in inches per blow for the last few blows.
W = weight of pile in pounds.

In view of the fact that experience is the chief factor in specifying "set," one has to be very cautious in relying on any formulæ connecting "set" with safe loads. Except in exceptional cases, however, the first formulæ (equations 3 and 4) coupled with the correct value of the factor p may be taken as fairly reliable.

Another formula frequently adopted for lighter piles is the "Wellington" formula, as follows:—

$$L = \frac{2wh}{S+C} \qquad (6)$$

C being a constant depending on the type of hammer used and frequency of the blows. This formula was recently applied by Mr P. Allan, M.Inst.C.E., in piling for wharves at Newcastle Harbour, New South Wales.

As the formula quoted by the British Steel Piling Company was published at the time the writer was driving 70 ft. long timber piles in the Thames estuary to a definite "set," he had the opportunity of applying both formulæ with the following results. The actual loads which these piles were called upon to carry was 80 tons each, and they were driven into Thames ballast with a final specified "set" of $\frac{1}{16}$ in. Applying the first formulæ (equations 3 and 4) the actual safe load worked out to 144 tons, and applying the second formula (equation 5) the actual safe load worked out to 153 tons. At first sight it appeared that the set originally specified had been more severe than was necessary, but it is as well to point out that, even where a factor of safety is already allowed for as in the above formulæ, a wide margin, where obtainable, should be adhered to, principally because so little of the theory of pile-driving has yet been developed by civil engineers.

The Society of Engineers published a very useful paper by Mr A. S. E. Ackermann, B.Sc., in 1919, entitled "Experiments with Clay in its Relation to Piles." The paper is of great value from the theoretical point of view, and it throws a good deal of light on previously little understood phenomena. It suffers, however, from the following defects: (*a*) The experiments are confined to those made with models; (*b*) loads were applied by continuous pressure and not by impact; (*c*) the experiments were restricted to clay.

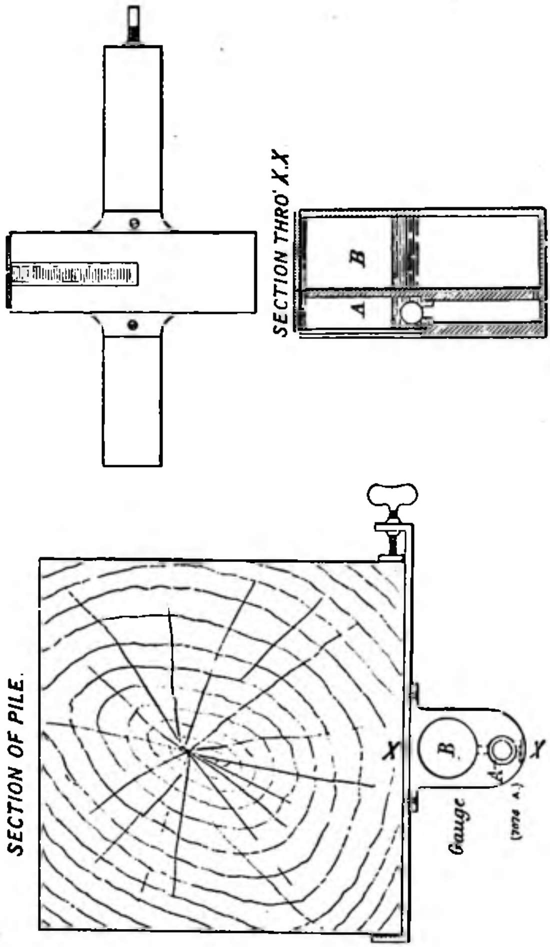
Briefly, in this and in a subsequent paper, Mr Ackermann establishes the fact that at a critical pressure, varying with the nature and percentage moistness of the clay, the latter behaves physically as a liquid. This pressure he terms the "pressure of fluidity," and doubtless it explains the large amount of recoil shown by piles driven into London clay and similar foundations.

The formulæ quoted in this chapter must not be applied to quickly operating single-acting steam or pneumatic hammers or to double-acting hammers of the M'Keirnan-Terry type. These types are of doubtful value in heavy pile-driving such as is experienced in marine work, and, in any event, more as yet has to be learnt about the "equivalent" performance of these hammers before anything in the nature of formulæ can be applied to them. In considering the question of permitting the use of such hammers for pile-driving to form foundations for a large works under construction in 1919, where the permanent piles had to carry very heavy loads indeed, the writer came to the conclusion not to advise the use of these hammers unless the contractor was prepared to carry out an elaborate series of comparative tests between such hammers and the more common, though perhaps old-fashioned, "clean-drop" falling weight type.

One of the principal difficulties experienced by resident engineers on public works is that of determining the actual set of the pile per blow without incurring the cost caused by the employment of a skilled observer and the delay referred to above of constantly stopping pile-driving for the purposes of observation.

It is a curious fact that in such a common operation as pile-driving so widely applied and utilised throughout the

world, no serious attempt has apparently been made to design a suitable gauge for the purposes of making such measurements. This has probably been due to the obvious



FIGS. 32, 33, and 34—Suggested Design for Penetration Gauge.

difficulties of subjecting a delicate instrument to the severe shock and jar of pile-driving and to the difficulties of obtaining a suitable fixed point against which to measure the downward movement of the pile per blow under vibratory conditions.

The writer made one attempt to design such a gauge and to measure penetration by means of a single attachment to the pile, relying on the inertia of a liquid to record penetration.

The original design evolved from this idea is shown in Figs. 32, 33, and 34, and consisted of a reservoir B communicating with an indicator column A, fitted with a ball valve. The intention was to fix the whole gauge rigidly to the pile, as shown in the figures. Under the impact of driving it was thought the body of the gauge would descend while the liquid contents would remain momentarily suspended, the liquid passing upwards in both chambers, but being trapped in chamber A—the amount of movement being read on the graduated scale. The second chamber B was intended to act merely as a reservoir for supplying liquid to chamber A.

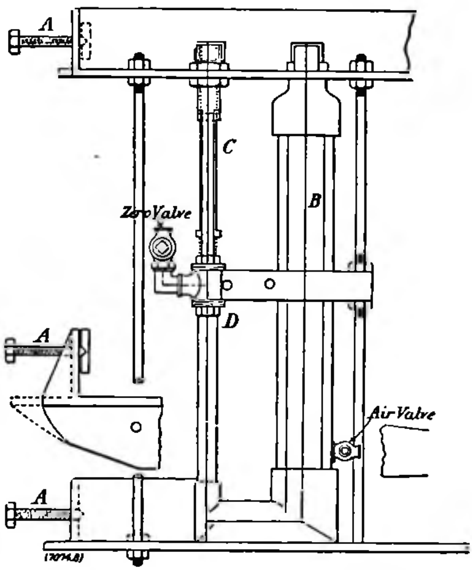


FIG. 35.—Gauge for Measuring "Set."

By the courtesy of the Empire Paper Mills Ltd., trials with such a gauge were made by Messrs P. & W. Anderson Ltd., contractors, at Greenhithe, Kent, in February 1921. These trials were conducted by Mr Percy Neate, M.I.Mech.E., who re-designed the gauge so as to correct the obvious faults in the original design. In the experimental gauge a very sensitive disc valve replaced the ball valve shown, and the supply chamber was fitted with a non-return air valve to enable both columns of

water to be in exact balance before the delivery of the blow from the pile-driving hammer. This gauge is illustrated in Fig. 35, in which AA represent the clamps, and B the reservoir chamber as before. In this case, however, the "trapping" chamber is merely a piece of steam pipe D with the sensitive non-return valve at its head and an ordinary gauge glass C for the purposes of observation. The principle of this gauge as actually made was identical with the first suggestion.

FIG. 36.—Pile-Driving Gauge under Test.

The trials were disappointing, owing to the heavy weight and consequent high relative inertia of the gauge fittings, weighing in all 59 lbs. The action of the gauge, however, was definite and well marked, the actual readings being inconsistent owing to movement which took place between the gauge and pile head, which movement would not have occurred had a lighter gauge been used with a more satisfactory attachment to the pile. The gauge was found to work with or without an air balance, and presents possibilities of development.

The view (Fig. 36) shows this experimental gauge under test. The penetration of the pile was observed from adjacent firm ground by means of a dumpy level, and the actual test sheet obtained is given in the subjoined Table VIII.

The recording chamber consisted of a gauge glass in the case of this experimental gauge. The inconsistency of the figures is apparent. The gauge, however, was designed to magnify the record by 5 to 4 by constricting the internal diameter of the recording chamber. Unfortunately, continual movement was observed between gauge and pile, and the only observations taken under anything like rigid conditions were blows Nos. 33 to 39 inclusive, which are fairly consistent with the observed penetrations.

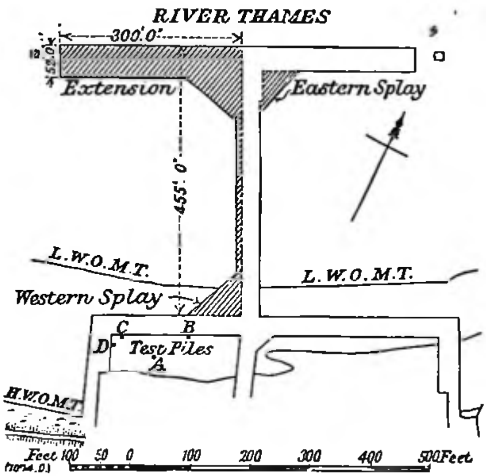


Fig. 37.—Site of Pile-Driving Tests, Greenhithe.

The site on which the tests were taken was unique in respect of the facilities afforded for observation. The author's firm (Messrs Carey & Latham), being at the time responsible as consulting engineers for carrying out several contracts for the Empire Paper Mills, were desirous of ascertaining the nature of the ground behind an existing wharf with a view to extending its capacity landwards.

These new works are shown in the plan (Fig. 37), and the position of the test piles is shown also. Piles C and D afforded singularly good opportunities for observation, being driven practically right against a permanent and rigid existing structure from which accurate measurements of final sets per blow could be taken.

Attempts were made to obtain an autographic record of

TABLE VIII.—TEST ON INERTIA GAUGE. NEW JETTY CONTRACT. EMPIRE PAPER MILLS LTD., GREENHITHE.

No. of Blow.	Penetration Recorded by Gauge.	Penetration Recorded by Actual Observation.	Remarks.
	In.	In.	
1	$1\frac{1}{2}$	$1\frac{3}{16}$	Air balance.
2	$1\frac{3}{4}$	$1\frac{1}{16}$	,,
3	$1\frac{15}{16}$	1	,,
4	$1\frac{15}{16}$	$1\frac{1}{16}$	,,
5	2	$\frac{15}{16}$	,,
6	$1\frac{11}{16}$	1	,,
7	2	$\frac{15}{16}$	,,
8	$1\frac{3}{8}$	$\frac{7}{8}$	Gauge slack on pile head.
9	$1\frac{3}{16}$	$\frac{15}{16}$	,, ,,
10	$1\frac{1}{8}$	$\frac{15}{16}$	,, ,,
11	$1\frac{1}{2}$	$\frac{7}{8}$	,, ,,
12	$\frac{15}{16}$	$\frac{13}{16}$	No air balance.
13	$\frac{1}{2}$	$\frac{7}{8}$	Gauge still slack on pile head.
14	$\frac{3}{8}$	$1\frac{1}{4}$	,, ,,
15	$\frac{1}{2}$	$1\frac{3}{8}$	,, ,,
16	$\frac{3}{4}$	$1\frac{9}{16}$	,, ,,
17	$\frac{3}{8}$	$1\frac{5}{16}$	No air balance; gauge still slack on pile head.
18	$\frac{1}{8}$	$1\frac{5}{16}$	,, ,,
19	$1\frac{11}{16}$	$1\frac{5}{16}$	,, ,,
20	$1\frac{13}{16}$	$1\frac{1}{16}$	,, ,,
21	$2\frac{3}{8}$	$1\frac{1}{16}$	Attempt made to re-attach gauge rigidly to pile.
22	3	$\frac{13}{16}$	,, ,,
23	$3\frac{5}{16}$	$1\frac{5}{16}$	,, ,,
24	$3\frac{5}{16}$	1	,, ,,
25	$2\frac{13}{16}$	$1\frac{1}{16}$	,, ,,
26	$2\frac{5}{8}$	$1\frac{1}{8}$	,, ,,
27	$2\frac{13}{16}$	$1\frac{1}{16}$	,, ,,
28	$3\frac{3}{16}$	$1\frac{1}{8}$	,, ,,
29	3	$1\frac{1}{8}$	,, ,,
30	2	$\frac{9}{16}$	,, ,,
31	...	...	Diagram taken.
32	...	...	,,
33	1	$1\frac{1}{16}$	Air balance.
34	1	1	,,
35	1	$\frac{15}{16}$	,,
36	$1\frac{1}{8}$	1	,,
37	$\frac{15}{16}$	1	,,
38	$\frac{9}{16}$	$\frac{9}{16}$	,,
39	$\frac{5}{8}$	$\frac{1}{4}$	Gauge loose.
40	$\frac{1}{2}$	$\frac{1}{4}$	,,
41	$\frac{15}{16}$	$\frac{5}{16}$	,,
42	$\frac{15}{16}$	$\frac{7}{16}$	Air leak.
43	$\frac{15}{16}$	$\frac{7}{16}$	Gauge glass broke.

a blow in two cases (blows Nos. 31 and 32). This was done very simply by placing a smooth board across the face of the pile, rigidly resting it on the permanent works and passing a pencil slowly across its extremity, and against the pile during the blow. The first attempt failed owing to bad timing, but the second was curiously successful, and is reproduced in Fig. 38.

This diagram was taken by Mr Neate, and has been the subject of some speculation. It most emphatically shows that initial penetration in this case largely exceeded the final set. Mr Neate's view is that the first upward sweep of the curve represents the actual dip or drop of the pile head under the blow, the vibratory periods near the crest movements caused by the bounce or rebound of the monkey, and the final leg of the curve the recoil due to earth pressure exerted against the shoe of the pile by the violently disturbed strata beneath it. This view is in all probability correct, but the curve may be of interest to all civil engineers, and is worthy of discussion.

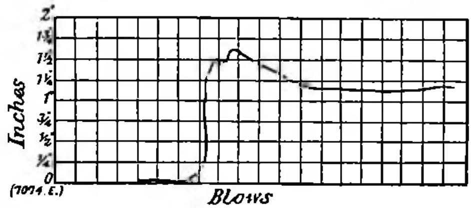


FIG. 38.—Autographic Diagram of Pile-Driving in Clay.

At the request of the Council of the Institution of Civil Engineers, the writer introduced the subject of inertia gauges in relation to pile-driving to the Engineering Conference of 1921, and the discussion which followed he considers to be of value, as also were two other inertia gauges exhibited at the conference. These gauges were respectively exhibited by Mr A. S. E. Ackermann, Assoc.M.Inst.C.E., and by Mr J. W. Fergusson. In both cases the idea of an inertia gauge to record pile sets had been obtained from the writer, but the gauges themselves were substantially different in design. The Ackermann gauge is based on utilising inertia, but no liquid is employed. The gauge consists of a weight suspended by a helical spring. The top end of the spring is either rigidly attached to the pile or in the improved design

suspended from a guide in a non-return ratchet, and the weight hangs freely. Under impact, the pile is driven downwards and the weight momentarily remains in its original position. The closure of the spring, therefore, is a direct measurement of the movement of the pile. Mr Ackermann's gauge has been highly developed in many details. An autographic curve is obtained by a pencil attachment, and a horizontal movement is given to the recording chart. The net result is an autographic curve similar to that shown in Fig. 38. Mr Ackermann has kindly supplied the author with a record of his experiments, and has given permission for the appended figures to be published. The results are given below, and the increasing error as the penetration increases is a noticeable feature.

TABLE IX.—SUMMARY OF RESULTS OBTAINED WITH THE ACKERMANN GAUGE

Date.	No. of Experiment.	Actual Set of Pile in C.M.S.	Set Recorded by the Autographic Curves in C.M.S.	Remarks.
1921.				
25th June - - -	267 (*b*)	2.45	2.45	Max error – 5.6 per cent. too little.
25th June - - -	267 (*c*)	2.70	2.55	
25th June - - -	267 (*d*)	2.46	2.40	...
25th June - - -	267 (*e*)	2.53	2.70	Trip gear failed.
25th June - - -	267 (*f*)	2.43	2.35	...
26th June - - -	268 (*a*)	2.36	2.36	...
26th June - - -	268 (*b*)	2.25	2.25	...
26th June - - -	268 (*c*)	2.20	2.15	...
28th June - - -	269 (*a*)	2.00	2.00	...
28th June - - -	269 (*b*)	1.90	1.90	...
Totals - - - -	...	20.75	20.41	Mean error – 1.64 per cent.
Total error - - -	...	...	– 0.34	
28th June - - -	270 (*a*)	3.80	3.60	...
28th June - - -	270 (*b*)	4.00	3.65	...
28th June - - -	270 (*c*)	4.00	3.80	...
Totals - - - -	...	11.80	11.05	Mean error – 6.35 per cent.
Total error - - -	...	...	– 0.75	

CHAPTER V

THE CONSERVANCY OF MARSH LANDS[1]

Sewer Commissioners—Standard Type of Clay Embankment—The Manner in which Failure may take place—Examples of Failure —Tidal Hydro-Electric Problems as applied to Riparian Marsh Lands.

THE protection of low-lying marsh lands against inundation by sea or river is of importance. Where there is no legal liability to maintain a sea or river wall, and the problem is one solely to be decided by landowner or tenant, the question of conserving land is frequently reduced to the consideration of the expenditure involved in reclamation or maintenance works. In the former case, where land has been allowed to "go to sea," it is seldom that the problem can, in this country, be economically solved, *i.e.*, in other words the ultimate value of the land reclaimed is too low to justify the expense of reclamation. There are sometimes, however, other considerations, æsthetic or legal, which overrule this consideration; but, in general, the wise policy is to "maintain," and therefore not to have to "reclaim."

Many marsh areas are under the control of Sewer Commissioners. Where this is the case, though sometimes heavily rated, sea and riverside properties are fairly immune from inundation. Breaches, however, have occurred from time to time in embankments maintained by Sewer Commissioners, *e.g.*, in the Parish of Stone, Kent, in 1896, and the same year on the Fobbing Levels, Essex. The administrations of these independent bodies of Commissioners are by no means standardised, and their boundaries are somewhat haphazard. It is to be hoped, however, that under the wide powers of the Drainage Act of 1918, areas may be merged, new areas

[1] Including an article in *Engineering*, 6th January 1922.

created, and powers vested in County Councils in such a way as to ensure the consistent protection and drainage of all low-lying lands in Great Britain.

At the time of writing, certain definite steps have already been taken by the Board of Agriculture and Fisheries, notably in respect to the Rivers Ouse, Crouch, and Blackwater in East Anglia and on both banks of the River Colne.

Clay Embankments.—The type of embankment on the Essex Flats is more or less standardised. A dyke is cut at the back of the bank, into which the main outfall channels are carried, and these discharge the drainage of the marshes through sluices in the wall; these sluices are provided with tidal flaps at their seaward end. The level of the marshes generally averages from 7 ft. to 10 ft. below high-water level, and the back slope of the embankment, though supposed to be 1½ to 1, is more generally 1 in 1. The crest of the bank averages about 3 ft. wide, and the outer slope starts at 1½ to 1, either until it meets the natural protective saltings or an easier gradient (about 5 to 1) leading on to the natural foreshore. The sea or river face of the wall is pitched with block chalk or Kentish ragstone, or both, sometimes from the crest to the toe, and sometimes only from high-water level to the toe. Where the protective saltings are wide, and the sea or river face of the wall consequently short, this protective pitching is sometimes omitted; where no saltings exist, the toe of the slope is protected—either by light elm piles or random stonework dumped in mounds on the foreshore. Rough attempts at groyning are also sometimes carried out. It is useful to quote the above typical section, because similar protective works are also found on the Kentish coast, on the Norfolk and Suffolk rivers, on the banks of the River Severn, in Ireland, and elsewhere.

Breaches.—The greatest threat to which this class of protective work is exposed is that of a breach. This form of failure may occur in any of the six following ways:—

A. By damage to pitching, owing to the impact of waves.

B. By subsidence of the wall forward, owing to scouring action below high water mark.

C. By subsidence vertically downwards.

D. By subsidence of the wall landwards. .

E. By constructional work such as wharves, factory foundations, or other riverside works.

F. By erosion on the face of the bank, caused by direct wave action.

These classes of failure are dealt with *seriatim* below.

Class A.—It will be found that damage to pitching is of very common occurrence. In the winter of 1912, during a gale from the south-east, considerable quantities of pitching on the north bank of the River Thames were dislodged and breaches in the embankments threatened. Only prompt action by the Commissioners' surveyor on this occasion saved the situation. Damage is progressive in the following order. After the pitching has been dislodged, wave action or scouring action of the tide rapidly takes out the clay hearting from behind; the crest of the wall then subsides, and the next high tide flows over the crest thus lowered; the first trickle of water quickly becomes a torrent, and weir action takes place inside the wall. Although the water may be quite shoal outside the bank, it is not at all an uncommon thing for the digging action of the water to bore a depression in the marshes 40 ft. or 50 ft. deep, into which the adjacent banks then slide. A single tide is sufficient, in some cases, to complete the damage. This digging or boring action of the water renders the repair of such a breach most difficult. To repair a breach which is only threatened in this way is an easy operation. Where the evil has been allowed to go on, the cost of reparation, however, becomes extremely high. A breach of this kind was recently closed on the River Crouch, reclaiming an area of about 300 acres. The work was carried out by unemployed labour, and cost about £27,000. This perhaps is not quite a fair figure to quote, as unemployed labour is always expensive. The engineer retained in this case estimated the cost of the work under ordinary contract conditions to be approximately £9,000, but even this latter figure was, in the instance referred to, disproportionate to the value of the land reclaimed.

The author reported on such a breach on the River Deben, in 1913, but the cost of closing it was out of all proportion to the value of the land.

Another example of this class occurred at St Osyth, Essex, in 1920. In this case there were some 250 acres of low-lying land threatened with inundation. The two photographs reproduced illustrate the condition of this frontage: 350 lineal ft. were similar to the photograph, Fig. 39, and 1,400 lineal ft. like the photograph, Fig. 40, at places even worse owing to the complete demolition of the timber facing by decay preceding wave action. Figs. 41 and 42 give the respective typical cross sections, and Figs. 43 and 44 the proposals for refacing the embankment in reinforced concrete. The site is at the estuary of the River Colne where it joins

FIG. 39.—St Osyth Sea Defences.

the sea, and the tidal conditions were of special interest, failure being due to a combination of scour and wave action. This case was also a classic example of the necessity of determining the economic relationship between the expenditure necessary to reinstate the wall and the value of land protected, which in this case and at that time probably did not exceed £20 per acre.

Class B.—An example of Class B was that of a slip occurring to an embankment on a property at Purfleet. The owners in this case constructed a jetty, with its berthage parallel to the river bank. The approach works of the jetty were carried on reinforced concrete columns 5 ft. in diameter,

which columns deflected the set of the ebb tide inwards, onto the river bank. This action scoured away the foreshore and disclosed a very unstable formation, known locally as "moor-logg"; this formation, sliding forward into the river, caused the embankment to move forward with it. Under an old Act, the wharfingers sought to recover part of the cost of reparation from the adjacent landowners, and an action in respect of this was heard before the Official Referee in the High Courts in 1908, when a verdict was given in favour of the plaintiff. As a consequence of this judgment, the landowners concerned are held liable to meet all such

FIG. 40.—Another View, St Osyth Sea Defences.

charges which are likely to accrue in the future. In view of the mode of reparation adopted the trouble is likely to be progressive, and the position of the contributory landowners is, therefore, not an enviable one. An extensive marine survey was undertaken in connection with this action, the result of which, in the author's opinion, conclusively proved considerable scouring action to have taken place (see Fig. 70, p. 134). The statement, therefore, put forward by the defendants, though not accepted by the Court, would appear from the engineer's point of view to be correct, namely, that the only possible method of arresting the movement of the bank was to carry out protective works on the

foreshore in front of the wall, at or near low-water mark, and by the grading and facing of the river bank from that level backwards and upwards to the crest of the wall.

Class C.—This class of subsidence in clay embankments involves a most careful investigation as to the nature of the underlying strata. It is sometimes due to pumping operations and occasionally due to dredging. Pumping

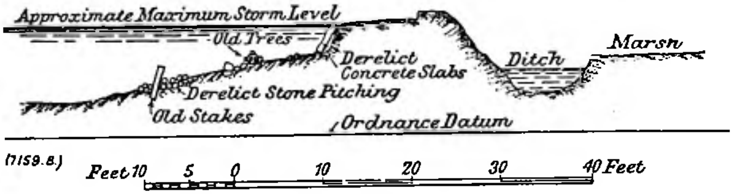


FIG. 41.—Cross Section of Fig. 39.

operations in connection with the Severn Tunnel had resulted, just prior to the war, in the flooding of over 200 acres of marsh land in Monmouthshire, owing to sinkage of land levels and reversal of drainage on the Caldicot Levels. In a recent case coming under the author's notice, dredging operations 4,000 ft. away from the river bank produced a settlement of 12 in. vertically downwards within a period

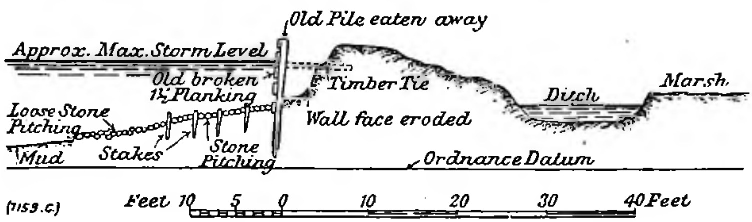


FIG. 42.—Cross Section of Fig. 40.

of eighteen months. Such an occurrence is a serious matter where low-lying lands are involved; a few inches will suffice to endanger the safety of protective embankments. Directly settlement is suspected, periodic observation should be undertaken at once at regular intervals in order to establish evidence between cause and effect; borings may also sometimes have to be undertaken. Very little can be done to

arrest this class of subsidence, as the marshes and foreshore usually accompany the wall in its downward movement. If, however, it can be established that no appreciable settlement had occurred over a period of years prior to the dredging or pumping operations, the parties responsible for the latter can usually be restrained by legal process.

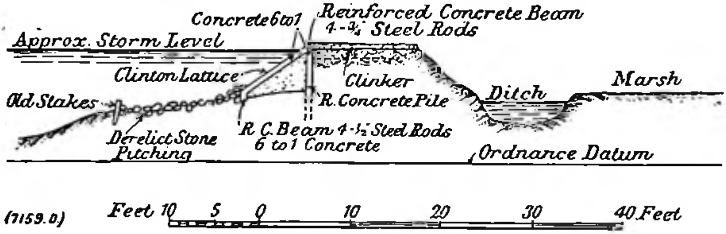


FIG. 43.—Proposal for Dealing with Cross Section Fig. 41.

One exceptional case of subsidence vertically downwards occurred on the sea-wall at Leysdown, Isle of Sheppey, Kent, immediately following the Armistice terminating the Great War. During 1917 and 1918 the Air Force stationed in the island at Eastchurch and Leysdown utilised the foreshore in front of this sea-wall as a practice ground for bombing

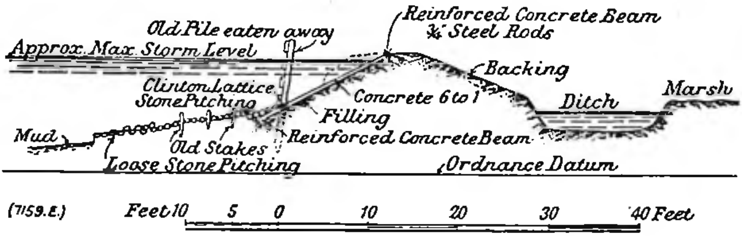


FIG. 44.—Proposal for Dealing with Cross Section Fig. 42.

from aeroplanes with high explosives. The foreshore here consists of London clay, and is over a mile in width at low water. Practice targets were anchored on the foreshore, and a continuous rain of bombs fell on it for two years. Frequently bombing was continued when the foreshore was dry, causing craters some 10 ft. or 12 ft. deep within about 300 yards of the apron of this wall.

The profile of the wall itself is shown in Figs. 45 to 47, and it consisted of London clay pitched on the surface with stone and grouted in with sand cement.

FIG 45.—Leysdown Sea-Wall.

An approximate section is shown in Fig. 48. The normal expenditure on this wall for many years had not exceeded

FIG. 46.—Leysdown Sea-Wall.

£100 per annum, but, as the result of bombing, the expenditure in two years rose to over £3,000. Failure

was caused apparently by the radial impulse of explosions passing through the fabric of the wall and causing the clay to settle in places. The extent of this settlement reached

FIG. 47.—Leysdown Sea-Wall.

3 ft. in the worst cases, and the surface of the apron collapsed into the cavities so formed.

Fig. 49 gives a close view of the pitching which the Air Ministry maintained was in bad condition, and the prime cause

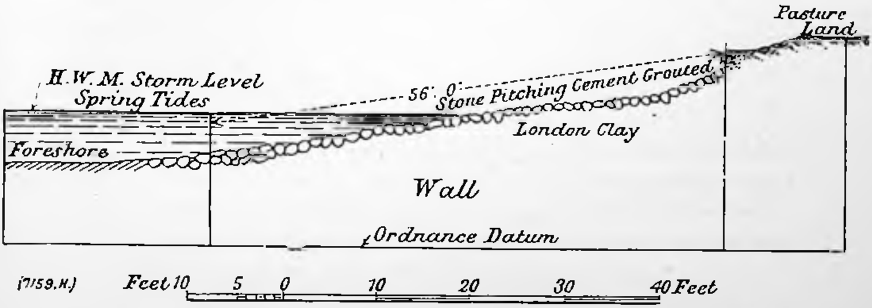


FIG. 48.—Profile of Leysdown Sea-Wall.

of collapse. While, however, this particular section of wall admittedly required the usual attention which maintenance of this class of wall calls for, there was, in the author's opinion, no evidence that its condition had in any way been abnormal

prior to the commencement of bombing practice on the adjacent foreshore. The curious fact about this failure was that no cracks occurred on the surface of the wall, nor was there any evidence of clay from behind the pitching being washed out, failure coming suddenly in the form of collapse of the pitching into the cavities as above described.

There is little doubt that this failure was directly attributable to "wave action" in the deep strata of London clay, and the recent experimental investigation made by Mr A. S. E. Ackermann B.Sc., indicates that the detonation of high explosives momentarily reduced the clay to a fluid state

FIG. 49.—Damage to Leysdown Sea-Wall.

owing to the high local specific pressure exerted radially through the strata.

Class D.—The class of failure referred to under D is uncommon, and the author in his experience has had only one such case to deal with. In this particular case the movement was arrested by an immediate dumping of material behind the wall, thus easing the slope and reducing the earth pressure. Where this form of slip does occur, it is usually after heavy rains, or due to the jamming or failure of sluices, permitting the entry of river or sea-water on each side. The marshes are then liable to become "soggy," and the movement starts by subsidence at the heel of the wall.

Class E.—Little can be said with regard to breaches threatened in the manner indicated under Class E, beyond noting the fact that the trouble usually arises from the overloading of the wall in some form. Several instances of this have come under the author's notice, but as the problem in each case presents a different aspect, it is impossible to generalise. The stability of most foundations on marsh lands depends on the even distribution of their loading, and failure to comply with this physical requirement has frequently been the cause of considerable settlement in buildings and adjacent earthworks.

Class F.—The phenomenon met with under F is of frequent occurrence. One case with which the author dealt was in connection with the protection of a considerable stretch of river embankment on the north side of the River Alde. For some time, up to the year 1910, owing to wave action during high tides, this embankment had been rendered nearly vertical on its river face, and in some places it was actually of a concave formation. The bank was first faced with carefully selected clay puddle to an even slope of 1 in 1½; some old timber piling at the foot of the wall was cut to a level uniform with the foreshore and then backed with mass concrete. Behind this, in place of the usual chalk and rag-stone pitching (in this district known as stud and plaster work), keyed concrete slabs were laid on the De Muralt system. It was found that this system readily accommodated itself to the curves and re-entrant angles of the river bank. It is more durable and scour proof than ordinary pitching, and in the case quoted met with complete success. In view of the economical character of the work, the owner of this property extended the facing down river in 1911, and again in 1912; in all, there are now some 2,000 lineal ft. of this bank so protected, and it forms a useful object lesson.

De Muralt and Decauville Facings.—There are apparently only two effective substitutes for chalk or stone pitching in vogue; one is the De Muralt system referred to above, which has been employed with success on the banks of the Rivers Thames,[1] Medway (see Fig. 26, p. 43, and Fig. 27, p. 44), Crouch, Alde, and Blyth. The second substitute is the

[1] *The Engineer*, 15th July 1910, p. 71.

Decauville system, of which the author has no personal experience; in cases where earth pressure from behind is encountered, it would appear to serve a useful purpose, more especially where the banks are steep.

Reparation of Breaches.—Where complete breaches in clay embankments of the kind under consideration have occurred, their repair is a most difficult question. All sorts of ingenious devices have been suggested and tried, but have generally been conspicuous by their failure. Measures such as the sinking of barges in the gap, or attempts to cross the breach by sheet piling alone, are generally to be condemned. In many cases they serve only to widen and deepen the breach as work progresses.

A sound policy to pursue is often deliberately to remove the banks for some distance on either side of the breach, which permits of an easy influx and efflux of tide, in place of the gorged current through the original breach. The pot-hole cut by the inward rush of water can be filled from hopper barges, or by dumping material overside from other floating craft at or near high tide. The bank can then be reconstructed in upward steps of 1 ft. or 18 in. at a time over the whole width of the widened gap, due precautions being taken to prevent the removal of material by each successive tide. Another, although more expensive alternative, is that of a temporary timber dam, driven seawards of the gap and provided with an efficient sluice, or sluices, so that the flooded areas can be drained and reparation take place behind the protective dam.

A common operation resorted to in such cases is that of constructing an inset clay embankment, effecting junctions with the old wall on either flank of the breach; indeed, owing to questions of cost, it is this last alternative which is usually adopted. It is, however, unsatisfactory, because it creates awkward re-entrant angles to the alignment of the wall, which are likely sources of future trouble, while a certain area of land between the inset and original alignment of the old wall has to be permanently abandoned.

Tidal Hydro-Electric Problems.—At the time of writing, some amount of public attention has been called to the problem of utilising tidal power. As efficient turbines are

now procurable for operating under very low heads, the possibility of utilising otherwise waste marsh lands for tidal storage is of interest. The class of work involved by proposals for establishing tidal hydro-electric installations, whereby the great reserve of tidal energy may be converted to "the use and convenience of man," has always been a fascinating subject to engineers, but as no practical example on a large scale exists, the actual financing of such schemes has always presented difficulty. Many so-called tide mills have been in existence at different periods, notably at Bishopstone, near Newhaven; St Osyth, Essex; and on the River Rance, Brittany. They are all quite distinct from the hydro-electric installations of the American Continent, where the hydraulic "flow" is always in one direction, and where the civil engineer's province is restricted to the study of river discharges and the design of suitable head and tail races with their accompanying accessory works. Such installations depend primarily on the use of river discharge, and differ fundamentally from any attempt to "harness the tides." In this country the subject was brought forward prominently by Sir Eric Geddes in the House of Commons on 30th November 1920, when replying to questions concerning the then proposed establishment of a hydro-electric scheme on the River Severn. There appeared to be many, and perhaps insurmountable, objections to that particular scheme, and it was not truly "tidal" in that the source of power comprised two factors, viz.:—

1. The natural effluent due to the rainfall in the River Severn catchment area.

2. The tidal effluent due to the volume of tidal waters impounded twice daily behind the proposed barrage.

The first factor represented a pure river hydro-electric proposal, and the second a pure tidal hydro-electric proposal, and it is well clearly to differentiate between the two. The maritime engineer, if we properly understand the definition of his calling, has nothing to do with the first class of undertaking, but is intimately concerned with the second.

Early in 1921 the author was asked to investigate a proposition for establishing a hydro-electric installation in the Island of Mersea, Essex. The scheme was first looked askance at as being of a visionary character. The promoter

of the idea was not an engineer, and therefore naturally suffered from certain disabilities, including the want of a proper realisation of the fact that any hydro-electric scheme must of necessity be devised on the lines of continuous output,

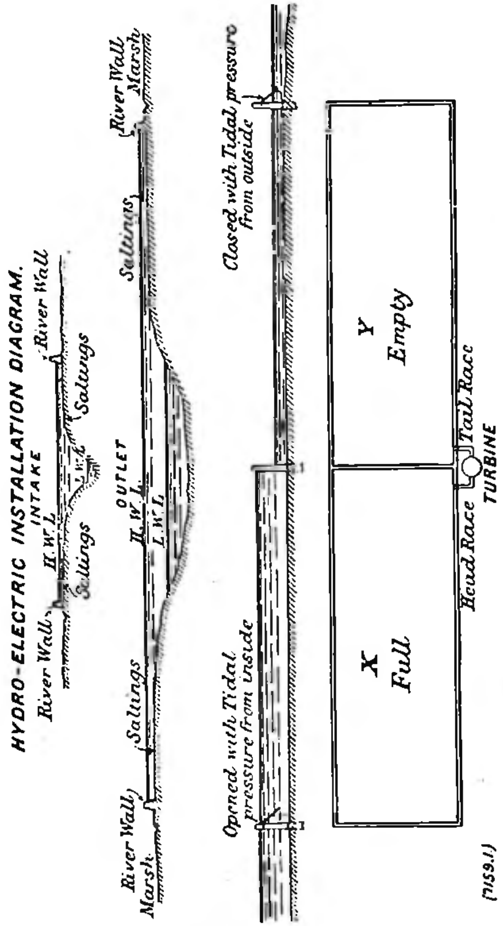


FIG. 50.—Diagram of Proposed Power Scheme, Mersea Island.

intermittent supply being useless except in those industrial centres where the supply of power, though intermittent, can be balanced to some extent at a central station. This latter possibility did not present itself at Mersea.

There was some difficulty in convincing the promoters

that, because 5,000 horse-power was running to waste daily, it might be difficult to "harness it," and even it out over the twenty-four hours! By chance the topographical character of the country, however, did lend itself to an enterprise of this

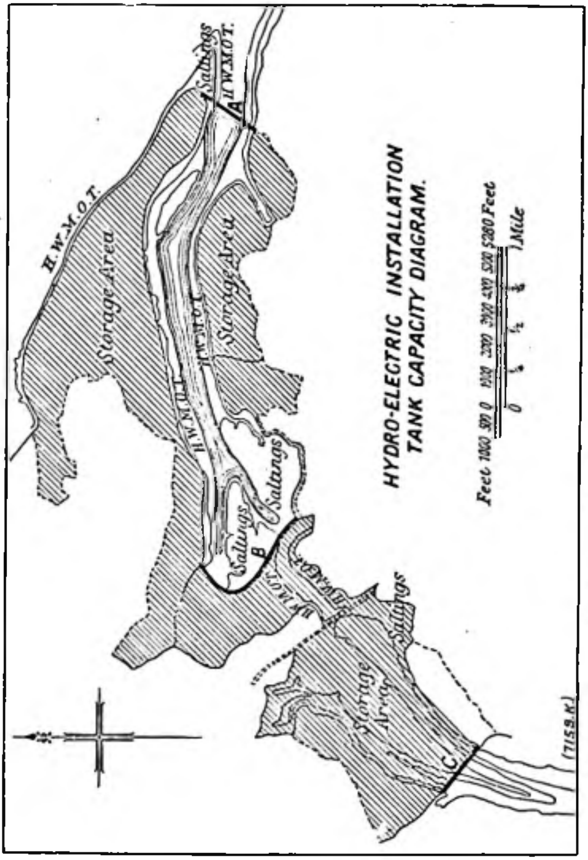


FIG. 51.—Plan of Proposed Hydro-Electric Installation, Mersea Island.

character. The diagram (Fig. 50) to some extent explains this when taken in conjunction with the plan (Fig. 51). The island—a reduced edition of the Isle of Wight—is separated from the coast line by two channels, the Strude Channel and the Pyefleet Channel. These channels penetrate low-lying marsh lands protected by clay embankments, and which, if

flooded by intentional breaches in the river banks, would give a potential head of water for power purposes. The range of spring tides was about 14 ft., and neaps some 4 ft. less, so that an effective working head of 5 ft. could be provided with horizontal type turbines over about half the tide cycle.

There being no natural site for a high-level reservoir for storage to cover the other half of the tide cycle, it appeared necessary to seek for a low-level empty compartment to take discharge during the otherwise inoperative period.

A hydraulic scheme was devised in accordance with Fig. 50, where Y, the empty compartment, was closed by tidal pressure from without, and was intended to provide the reserve capacity. By coincidence at low tide the Pyefleet Channel, according to the Ordnance sheets, showed a possible capacity as an "empty compartment," without dredging, of 36,000,000 cub. ft., and a general scheme on the lines of Fig. 51 appeared possible, though limiting the initial scheme to about 1,000 horse-power.

The works proposed were: (*a*) The construction of two barrages across the entrances of the Pyefleet and Strude Channels at A and C (Fig. 51); (*b*) the construction of a clay bank B dividing the supply and discharge areas (Fig. 51); (*c*) the construction of a power-house with the necessary turbines, generators, and accessories, and the necessary sluices and races.

The proposed scheme has not at the time of writing been carried out, but there is no doubt, with the demand for cheap power rapidly growing, tidal hydro-electric schemes in general will pass from the visionary to the concrete stage sooner or later. The investigation made at Mersea Island clearly shows, however, that suitable sites for tidal hydro-electric schemes are limited by the topographical characteristics of the coast line, and that prospecting for such sites will become quite a special branch of maritime engineering in the near future.

Another scheme of interest has recently been put forward by Brig.-Gen. Sir F. C. Meyrick, Bart. This proposes the use of both river and tidal waters at Milford Haven, Pembrokeshire, and was described and illustrated in the

Western Mail of 25th February 1921 and has been referred to in the *Times Engineering Supplement* of 20th May 1922.

Interesting examples of turbines working under low heads (down to 12 in.) already exist at the Chester Corporation power station, and on a small scale at Sir John Anderson's model farm at the village of Harrold, Bedfordshire.

The flooding of marsh lands for the purpose of establishing hydraulic power schemes will entail a very careful study of existing river and sea embankments, as the pressures on the embankments will be reversed, *i.e.*, they will be experienced on the backs of the river walls and be continuous, instead of being on the faces and intermittent with the rise and fall of tide. Suitable protection in the way of pitching or facing the land slopes, and in certain cases strengthening of the wall sections, will undoubtedly be found necessary.

The French Government, after a long study of the subject of utilising tidal energy for power purposes, has selected the estuary of the River Rance as being the most promising site for initiating a tidal hydro-electric enterprise, but here there is no question of utilising marsh lands for "storage purposes."

Amongst the many ideas for utilising the power of tides is that of a direct-acting machine, the first conception of which is the utilisation of the "up-thrust" of the tide, to raise a buoyant tank *against* the resistance of the drive of the machine during the rising of the tide, and to utilise the dead weight of such tank when flooded as it descends with the falling tide. Theoretically this is a continuous operation, but the maximum rate of rising and falling tides represents the maximum power developed, and this occurs at about half tide level on the ebb and flood, but for long periods in the neighbourhood of high and low tide such a machine would be practically inoperative, owing to the extremely slow rate of rise and fall.

The popular view is that such machines may be condemned alone on account of this great variation in power from the maximum obtainable to zero in the tide cycle. A little reflection, however, will show that this objection

can be removed in a very simple way by duplicating the machine, and giving the second unit a definite lag behind the first. This can probably best be explained by a simple diagram. The diagram (Fig. 52) divides the tidal range for convenience into four sections marked ABCD. Where the rising tide is referred to, these letters are given the plus sign, and on the falling tide the minus sign. The rising and falling tanks are shown as L and Z in the lower positions, and as Y and M in the upper positions. The cycle of operations may be described as follows :—

At the commencement of the rising tide the first tank is in the position Z. The tank has been run dry during the $-D$ period of the tide, having been held in this position

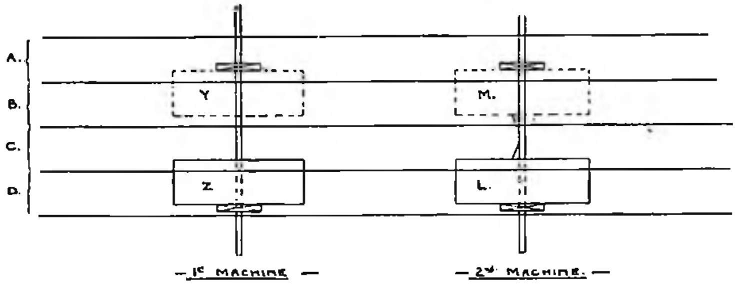


FIG. 52.—Tide Machine (Diagram).

by the stop shown on the guide. During the effective rise of tide, $+D$, $+C$, and $+B$, the tank will move upwards until it is in the position Y, as shown by the dotted lines. During the further period, $+A$, the tank is flooded, ready for its downward travel following the falling tide. For most of the period $+A$ and $-A$ the first machine, therefore, is not operating, but by the time it ceases to operate, the second machine, then in the position shown by the empty tank L, which has been held below water by an automatic catch, comes into operation. This tank is now released, and slowly rises during the tidal periods $+A$ and $-A$, reaching the dotted line position M, and becoming flooded and locked by a second automatic catch. The first tank still in position Y then commences to descend until it reaches position Z,

where it comes against a stop at the commencement of period $-D$. During period $-D$ and $+D$ the first machine is again inoperative, but the second machine comes into operation on releasing the catch under the tank, then at the dotted line position M, and this flooded tank finally completes the cycle of operations by its descent.

It is not within the scope of the present work to go into the question of designing such a machine, and Fig. 52 is purely diagrammatic; but it will be apparent to every engineer that *power* on this principle can be obtained in a continuous form, and that the transmission of such power from its source to a rotary movement is the principal mechanical problem involved.

It is therefore clear that the objection to a direct-acting tide machine is not that it cannot be made continuous in operation. The possibility of designing such a machine has purposely been discussed above, in order that the real obstacles to its employment may stand out more clearly from the other and older objection of non-continuous action.

The principal real objection is undoubtedly the fact that, owing to the limited rise and fall of tide, the operating tanks must be shallow in depth, and would have to be of excessive dimensions to secure any practical result in the way of power, while the structural difficulties and cost render the practical application of any such machine quite prohibitive, except on a very small scale.

This fact by no means rules out the possibility of establishing a purely tidal hydro-electric installation on the turbine system, but the argument is useful as eliminating one by one the impracticable ideas so often seriously advanced in regard to tide power machines.

The only practicable schemes are: (*a*) a high-level reservoir into which water can be raised by pumping, to form the stand-by for the inoperative periods; (*b*) a low-level reservoir for the same purpose. In general, the condition (*a*) would apply to hilly or mountainous country, and condition (*b*) to marsh lands, or low-lying country.

It is to be hoped that the foregoing notes will bring with them the realisation that a system of balanced reservoirs

offers the only practical solution for utilising tidal power, and that the general topographical characteristics of marsh lands renders them a favourable prospecting ground, since large areas of land at low valuation may be brought into a useful productive field by permanent flooding.

The journal, *Water and Water Engineering*, in its issue of 20th March 1922, p. 87, quotes an article from *La Nature*, entitled "Harnessing the Tides," and although this article has suffered somewhat in translation from the original French, it refers to tide machines, and also to the various systems of "Conjugate Basins," which it is suggested might afford practical solutions of the problem under given conditions.

One interesting form of hydro-electric scheme is the type which is not *purely tidal*, yet in a sense one on which the tides have a definite bearing. This class of scheme involves the construction of a barrage across a tidal river, utilising the river discharge as the source of motive power, but providing special measures to deal with the inoperative period towards high water. The River Severn scheme falls into this category, and the author had recently to report on another scheme of similar type in Pembrokeshire. This latter scheme was advanced by Brig.-Gen. Sir F. C. Meyrick, Bart., and was described in the *Western Mail*, 25th February 1921. The chief attraction of this scheme is the physical fact that a large portion of the county of Pembrokeshire drains into Milford Haven through the two Cleddau rivers.

Messrs Forbes and Ashford, in their book entitled "Our Waterways," published in 1906, give the aggregate catchment area of these two rivers as 196 sq. miles. The range of tide at Milford Haven is 24 ft. on spring tides and 18 ft. on neaps. The scheme briefly consists of constructing a definite barrage on the site of the present Neyland-Pembroke Ferry. This in itself has the advantage that it would enable the two present "dead-ends" of the Great Western Railway system to be joined; but there may be obstacles in the way of maintaining navigation facilities, as a certain amount of shipping at present passes up-river above this point, and compensation, or the alternative cost of establishing locks,

would be a definite burden to a power scheme, as indeed it is to many semi-tidal projects.

Nevertheless, the author is of opinion that the scheme is worthy, at least, of serious attention. On an assumed average rainfall of 36 in. per annum, and a "run-off" factor as low as 50 per cent., it is clear that some 3,500 horse-power is available if an average head even of 10 ft. can be maintained.

The tidal velocities at the site of the projected barrage are very high, and it may therefore be difficult to provide in addition the additional horse-power available and due to actual tidal effluent.

Sir Frederick Meyrick, working apparently independently *of other investigators*, has reached the notable conclusion of the necessity of a *low-level* reservoir to operate during the idle period, which, in the case of this scheme, only occurs towards high tide.

Immediately behind the site of the projected barrage is a small estuary known as "Cosheston Pill," and there appear to be no great engineering difficulties in providing an outlet to this natural reservoir independent of the barrage. This outlet would discharge into Milford Haven *below* the site of the barrage at low water, and the construction of a dam, sluices, and auxiliary plant at the narrow neck of the Cosheston estuary itself would provide a source of power for operating over an otherwise idle period.

To sum up the whole aspect of purely tidal hydro-electric schemes, it is essential to consider the question of first cost, and in this respect such schemes have the special advantage over purely river hydro-electric schemes of the fact that in general, the questions of financial compensation or the necessity of providing compensation water do not arise to the same extent, since the tides of the sea are apparently free and open to anyone to utilise.

The recent investigations (1921) of the Electricity Commissioners indicate that, unless the entire first cost of any hydro-electric scheme, tidal or otherwise, can be brought down to the basis of £75 per horse-power, it will be unable at the present time to compete against power schemes involving the use of fuel and prime movers as the source of power. This, of course, is a generalisation, since, for example, where

no coal or oil fuel whatever is available, a hydro-electric scheme might well offer the only alternative in a district where there is a large demand for power. With the rapid development of oil and spirit fuel distribution, however, this does not seem to offer a wide field, and engineers in future, therefore, will be compelled to uphold or condemn all projects to harness the tides solely on the grounds of first cost as compared with the cost of prime movers plus the capitalisation of the fuel bill.

CHAPTER VI

COAST DEFENCE[1]

Arguments for and against Groyning—Examples of Erosion and Coast Defence—Concrete Revetments—Classification of Coast Erosion—Weir Groynes—Stub Groynes.

THE subject matter of this chapter is confined to the consideration of preserving land from erosion by the sea in cases where the contiguous land level is above high-water mark. The question of protecting *sea-walls* is also included, irrespective of the land level behind, as the two problems have much in common.

It is generally recognised that in cases of coast erosion, natural defences, assisted by artificial means, are the most effective remedial measures when they can be secured, *i.e.*, the accretion of protective shingle or sand beaches by means of groynes, stockades, or other works. (See "Maintenance of Foreshores," Crosby Lockwood & Son, 1913.)

An interesting public inquiry on the subject of Coast Erosion and Groyning was held by Colonel H. Tudsbery, M.Inst.C.E., at Minehead, in May 1921, on behalf of the Ministries of Health and Transport, from which it appeared that some confusion of ideas on the subject of groyning had arisen as the result of an opinion expressed in the handbook referred to above. The author's experience since the year 1913 has, however, confirmed the opinion then expressed, and the following explanation may make the point clearer.

It is wise to regard groyning "as a last resort." The

[1] Reprinted from an article entitled "The Maintenance of the English Coast-Line," *Contract Journal*, 5th October 1921; and extracts from an article entitled "The Maintenance of a Coast Road," *British Builder*, February 1922; also from an article in the *Contract Journal*, dated 22nd March 1922, entitled "Groynes: Considerations of Design."

reason for this view is that the services of the maritime engineer are not usually called for until a case of coast erosion has become economically serious. Where agricultural land of relatively low value abuts on the coast, the slow erosion of the coast line, although perhaps a matter of national importance, makes little or no appeal to the landowner. When similar erosion, however, attacks a valuable property, either industrial or residential, the owner (or responsible authority) is faced with the necessity for immediate action.

One such example is that of the Barton Court Hotel foreshore, Hampshire. This may be regarded as a case where groyning would not have saved the situation in time. In this case erosion by the sea had caused consequent collapse of the cliff face, land drainage being an auxiliary cause. The cliffs at this point were some 60 ft. to 70 ft. high, and consisted of gravel and loam overlying sand and yellow clay, the whole cliff being supported on a base of slipper clay with a "dip" seawards. The physical cycle of erosion was as follows:—

The cliff face became overburdened with land water and fell away to the beach below. This debris slowly slid forward, and was washed away by tidal scour. After each successive landslide, high-water mark advanced seawards, and the landowners could not reconcile this fact with the equally obvious fact that serious erosion was taking place. On surveying the ground it was possible to gauge fairly accurately how long it would take to bring the cliff edge up to the building line. In 1913 there remained about 90 ft. in a direct line from the cliff edge to the building itself. Nothing, in fact, was done by the proprietors of the hotel on the score of expense, and groyning certainly would at first have done more harm than good. A system of groynes would have arrested erosion in the end, it is true, but not on the desired alignment. The natural angle of repose of this unstable cliff was apparent, and a protective wall at the toe of the cliff would have had to be carried out at the junction of the undercliff with the beach, which alignment was already as far back as it was safe to go to ensure the preservation of the property. It is admittedly a little difficult to make this point clear, but it is hoped that the diagram (Fig. 53) may do so.

This represents an arbitrary system of groyning. The dotted line shows high-water mark before the construction of groynes, which high-water mark, it is assumed, has been encroaching landwards in the direction of the right-hand arrow. The full black line represents the deflected high-water mark after the construction of the groynes, and after the latter may have been presumed to have brought high-water mark to rest. It is clear that the maritime engineer must have the distance "D" "in hand," and, if he has, and can afford *temporarily* to aggravate erosion at the base of the groynes, then groyning is usually quite a satisfactory remedial measure. If the distance "D" *cannot* be afforded, then groyning is likely to do more immediate harm than good.

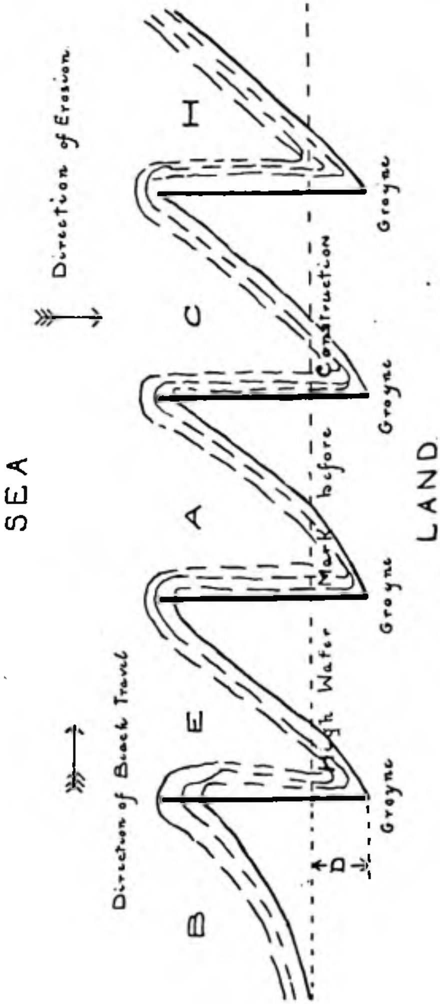


FIG. 53.—Theory of Groyning.

The reason why erosion occurs at the root of groynes is that the accretion of beach material on the side of the groynes facing the oncoming littoral drift causes a cascade action on the opposite or lee side, and this cascade of water

scoops out beach material and lowers the level of the foreshore. The effect can be minimised by the construction of groynes whose height can be adjusted as the beach level makes up, or by the construction of spur groynes.

It is interesting to note that the direction of beach travel is in the majority of cases in the direction of the flood tide on an open coast line. There are, of course, exceptions to this rule, and it is not generally true in the case of esturial waters. The cause of this phenomenon is the subject of much controversy, and many theories have been advanced. As, however, accurate observation of scour and current velocities should precede construction, this is a question, perhaps, of only academic and indirect interest to the practical engineer.

In one instance of coast erosion very definite data were recorded from 1899 to 1919. On the coast line between Minehead and Watchet, Somersetshire, a county road runs parallel with the sea for a distance of about 1,000 yds. in the vicinity of Blue Anchor railway station on the Great Western branch from Taunton to Minehead. In 1899, following considerable damage to the road, the late Mr Brereton, M.Inst.C.E., reported to the County Council, and as the result of his report the frontage was straightened to a fair line and the sea-wall rebuilt. This wall was in masonry and had a bull-nosed head, with an apron at the foot, and was sunk well into the foreshore. This wall is illustrated in the photograph shown in the frontispiece. At one spot, over a period of twenty years, the level of the foreshore dropped vertically by 13 ft., and so rapid had erosion become in 1921 that the foreshore had been eroded still further, giving a total vertical drop in level of the foreshore of approximately 15 ft.

Fig. 54, although not taken at the worst point, indicates the nature and extent of the erosion. The foreshore had originally been well covered with heavy shingle, and at the time of the writer's first inspection there still remained a certain quantity, but the lower part of the foreshore was badly denuded. The loam cliffs to the West showed evidence of rapid erosion, and a considerable bank of shingle had collected in the embayment thus formed. Current observa-

tions, and a careful inspection of the coast line to the East and West, confirmed the impression that beach material was

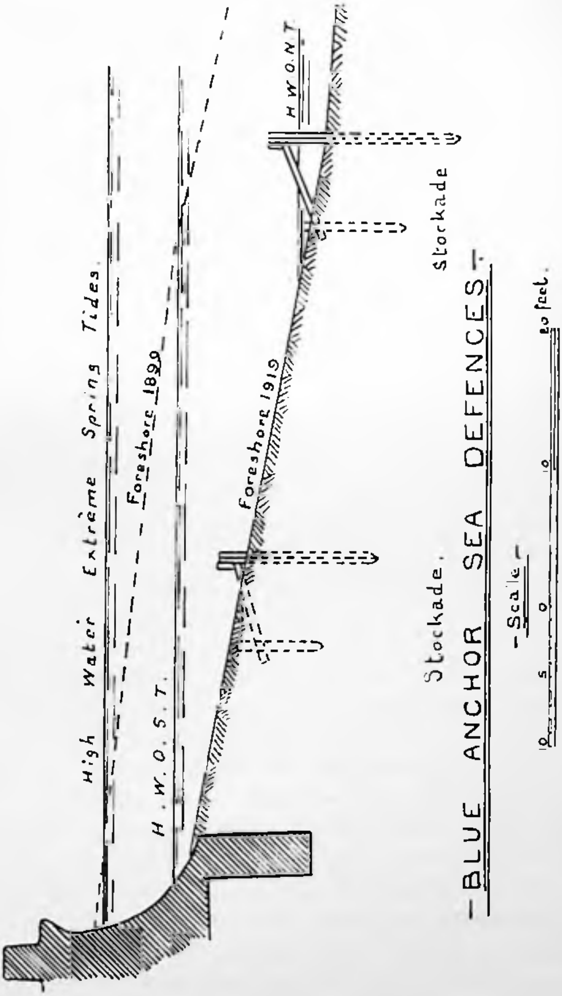


FIG. 54.

still travelling at this point from West to East up the Bristol Channel, though in reduced quantities.

Local geological conditions were of interest, as at the extreme Western end of the wall the "new red marl" occurs,

whereas the rest of the frontage consists of the Lower Lias deposits, appropriately termed "paper shales." The so-called shingle at this point is variable in size, running up to five or six inches in maximum diameter, and consisting principally of flat limestone fragments similar to those found in the adjacent shingle banks of Porlock Bay.

Recommendations were made to the County Council in 1920, but for financial reasons no extensive work could be undertaken until 1921, except certain temporary underpinning in concrete at the base of the sea-wall, a palliative which for several years had been adopted by the County Surveyor, Mr Edward Stead, Assoc.M.Inst.C.E., to prevent collapse of the sea-wall. The recommendations above referred to involved the construction of a groyne 1,000 ft. long at the Eastern end (subsequently reduced to 600 ft.), and two timber stockades, each 1,500 ft. long, running parallel to the sea-wall towards the Western end. The general lay-out of groyne and stockades is shown in Fig. 55, and was the result of a careful study of the site. The works were carried out by Messrs John Cochrane & Sons, contractors, Westminster.

The legal aspect of the case was curious. The road does not carry a great deal of traffic, but it appears that the County Council, as the Road Authority, are bound to maintain this road so long as it is not actually *destroyed* by the sea, *i.e.*, that they must constantly maintain it; but if, owing to heavy storm conditions or otherwise, the road should become completely breached, the Council are advised that there then would be no liability on their part to reconstruct. It was believed at the time the design was prepared that the groyne (Fig. 55) would have only a limited effect, *i.e.*, in protecting the road for only a limited distance towards Minehead. At the point where it was considered this effect would cease, the problem presented extraordinary difficulties. As the foundations of the wall holding up the roadway were actually exposed, including the shale foundations beneath, it was considered that groyning at this point, although capable of accreting shingle, would cause temporary erosion on the lee sides of the groynes of sufficient extent to cause subsidence of the road. After very careful consideration it was decided

that the case would best be met by the construction of two stockades parallel with the river wall from this critical point to the Western extremity.

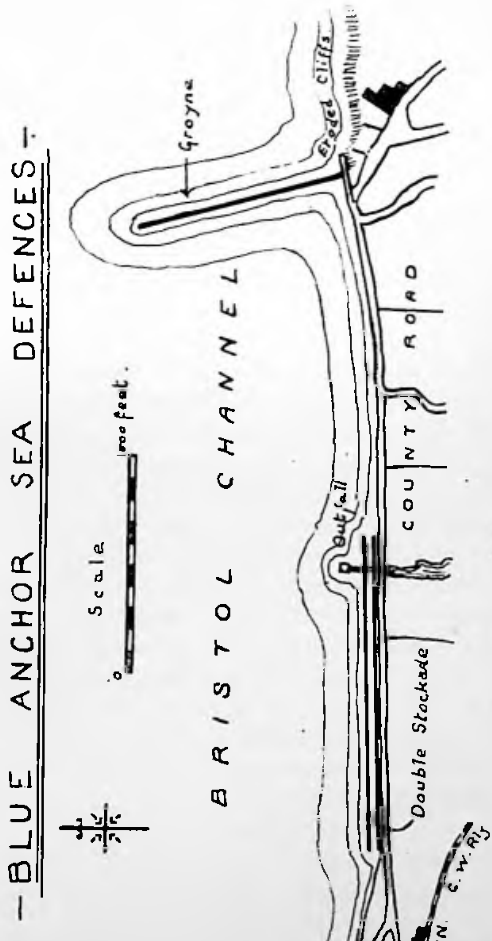


FIG. 55.—Plan of Blue Anchor Sea Defences.

The stockades consisted of main piles in pitch pine 10 in. by 10 in. section, with intermediate piles of 9 in. by 4 in. section bolted to continuous walings. The first stockade

has been constructed 20 ft. from the river wall, and a second stockade 50 ft. from the river wall.

It was thought that at some future time it might be desirable to close these stockades to retain any shingle collected, and Fig. 56, which shows these stockades, indicates the facility with which this operation could be carried out.

The contract work was commenced in July 1921, and the bulk of the works were completed in January 1922.

Fig. 57 shows the groyne at the point where the road runs down to the coast, and also illustrates the retaining wall of considerable dimensions which the adjacent landowner, Mr G. S. Lysaght, constructed to protect his own property and to complete the scheme of works adopted by the County Council. This wall forms the eastern bastion of the work. The stockade work was commenced at the Eastern and Western ends working towards the centre, and on 1st November 1921, before a gap in the stockades could be closed, very heavy North-Easterly gales took place and the road was again severely threatened, the foundations being badly undercut.

It was at first hoped that the whole of the stockade work could be carried out by means of pile-driving; but, after the original contract was let, further denudation of the foreshore exposed the shale formations. The total length of stockade was 3,000 ft., and ultimately about one-half of this length had to be constructed by bedding the piles in mass concrete, necessitating trenching in rock. Some idea of the difficulty of this operation is obtained from the illustration (Fig. 58), as blasting proved unsatisfactory.

Some idea of the effect of stockades on the accretion of shingle is given by the photograph (Fig. 59).

The total cost of the works, omitting the extension undertaken by the adjacent landowner, was approximately £30,000.

An instance of the erosion of low sandhills occurred on the links of the Royal Liverpool Golf Club, Hoylake, Cheshire. These links run down to the sea, with shallow sandhills abutting on the foreshore. One curious factor was that erosion was partly caused by tourists, who trampled down the sand slopes and caused initial disturbance of the frontage.

Fig. 56.—View of Stockade, Minehead-Watchet Road, Somerset.

FIG. 57.—Groyne and Sea-Wall, Chapel Cleeve, Somerset.

FIG. 58.—Rock Trenching, Minehead-Watchet Road.

FIG. 59.—Effect of Stockades on Accretion of Shingle.

In many spots sand "craters" existed and accentuated the tendency to erosion, which became more rapid as these craters were widened by wind vortices. The site was reported on by Mr A. E. Carey, M.Inst.C.E., in 1918, and stockade fences were erected and a labyrinth of gorse and brushwood cuttings dumped in the worst places. This measure has been entirely successful in arresting sand movement.

Another instance of serious erosion at the base of low sand dunes occurred at Burnham, Somerset, in 1920. In this case the landowners were recommended to establish two lines of low hurdles to trap the blown sand on the sea slopes and a double line of hurdles along the crest of the dunes at their lowest points. It was also suggested that the intelligent planting of marram grass on the dunes, as also gorse, sea buckthorn, *Salix repens*, alder, birch, etc., on the land slopes might prove advantageous.

A typical cross section of the Burnham foreshore is given in Fig. 60, showing the general proposal for "sand trapping" devices recommended. Those who are especially interested in the planting of vegetation in connection with coast defences are referred to the work on "Tidal Lands" (Messrs Blackie & Son, 1918), and to Mr Linn Chilvers, of Heacham, Norfolk, who has made a special study of this subject, and is a practical nurseryman.

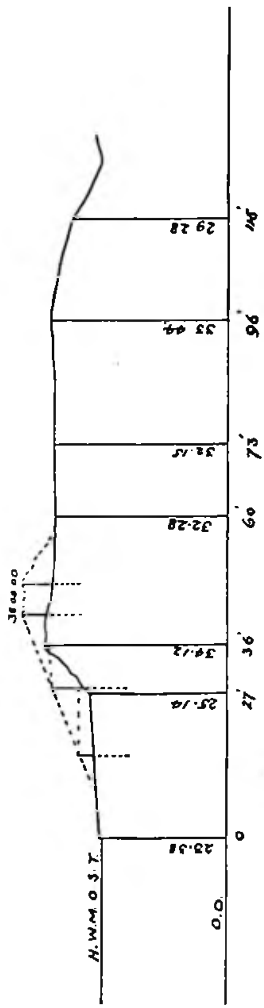


FIG. 60.—Sand Dunes and Foreshore, Burnham, Somerset.

The question of saving an exposed frontage by means of dumping beach material was successfully dealt with at Hove,

Sussex, and also to the east of Newhaven. Some years ago the foreshore in front of the Hove sea-wall as originally built was severely eroded, and the question of underpinning was under discussion. Locally, however, it was determined to try the experiment of bringing shingle from the West of Shoreham and dumping it before the wall. Some 20,000 cub. yds. were brought to the site in this manner after the erection of high groynes in 1883, and the shingle held by them has preserved the wall. At present the accumulation of shingle on the beach is level with the land behind the sea-wall. Mr Ellice-Clarke was the engineer responsible for this interesting experiment.

At Newhaven the extension of the sea-wall East of the harbour was necessitated by reason of the fact that the barrier of shingle had been neglected and the sea passed over the crest of it and lowered it to a dangerous extent. A concrete sea-wall was built along this frontage, and a series of groynes run out, shingle being brought from the West side of the harbour and dumped in front of the new wall. This operation was also successful, and has protected the frontage for over thirty years, prior to which large areas of land had been lost by erosion. The engineer for this work was Mr C. L. Morgan, M.Inst.C.E.

The coast at Southwold, Suffolk, was rapidly following that of Dunwich into the sea in 1905 (Fig. 71, p. 136), and the situation had become critical. The frontage concerned was about two miles in length, and the cliffs consisted of soft glacial deposit, offering little resistance to erosion by the sea. By means of the construction of a series of long groynes carried beyond low-water mark the destructive action was completely arrested. At one point opposite the Centre Cliff Hotel, where the exposed foreshore between high and low water was only 25 ft. wide, this foreshore has since become a broad sandy beach several hundred yards in width.

The De Muralt system is efficient in cases where sand accretion is desired, and has been effectively employed at Frinton, Essex, by Mr E. Montagu Bate on works under the direction of the author's firm. Fig. 28, p. 45, shows the erosion at the foot of the wall before the construction of a De Muralt apron, a cross section of which is shown in Fig. 29, p. 48. The

works were carried out in 1913, and have been entirely successful, the apron accumulating a natural revetment of sand which in a few months obliterated the new work.

The De Muralt system has certain advantages. It consists, in effect, of an articulated concrete revetment whose component parts are held down by an interlaced reinforced beam of "T" section. A lighter form of De Muralt facing can be employed on river banks subject only to scour or very slight wave action. Such facings have been used successfully at Tilbury (River Thames), Shellhaven (River Thames), Southwold (River Blyth), Aldeburgh (River Alde), and on the Medway at Chatham. A small area has also been laid down on the northern shore of the island of Foulness (River Crouch estuary). The earliest examples were those at Tilbury and Aldeburgh executed in the years 1910 and 1911, and all these defences have given satisfaction. (See issue of *The Contract Journal*, 30th October 1912.)

The De Muralt systems are well described in Mr Gerald Cases' book ("Coast Sand Dunes, Sand Spits, and Sand Wastes," St Bride's Press, 1914); also in Professor E. R. Matthews' work ("Coast Erosion and Protection," Charles Griffin, 1913).

The haphazard dumping of rock-like material at random for the preservation of a coast line or river bank is to be deprecated. Two instances may be quoted, viz., that of the waste material from the old copper works at Burry Port, where an artificial "cliff" so dumped caused serious erosion in 1913, near the entrance to Burry Port Harbour, Carmarthen, South Wales, and on the River Neath, Glamorgan, in 1919, in which latter case the natural régime of the river had undoubtedly been affected by the dumping of slag from the adjacent steelworks.

The foreshore of Chapel Cleeve, Somerset, is an interesting case of erosion. This property is owned by Mr G. S. Lysaght, and the foreshore forms part of a private estate. The coast line runs approximately East and West, and the Western extremity adjoins a heavy masonry sea-wall constructed about 1890 by the Somerset County Council. As far as one can judge, the alignment of this sea-wall with the loam cliffs of the Chapel Cleeve foreshore approximated

to the same line when the Council's sea-wall was constructed.

In 1921 these cliffs had receded to a point some 80 ft. in rear of the line of the wall. The beach, which consisted of heavy pebbles, has been graded by sea action, and the cliffs are rapidly wearing away. The travel of the beach is in the direction of the flood tide, and the re-entrant angle between the two properties was the site of considerable erosion. Recommendations for preserving this foreshore were made. These included cutting away certain old timber facings and pilings which had been undercut, and the construction of a revetment in reinforced concrete.

The site, however, is one of the beauty spots of England, and this proposal was thought to be unsightly, and an alternative design of a vertical concrete wall was finally chosen. Fig. 57, p. 104, shows this wall under construction, and Fig. 61 shows a section of the wall as now constructed. This wall section is of interest as running for most of its length at *right angles* to the beach line. The section of the wall changes at every foot from a wall some 28 ft. in depth to a low wall not exceeding 9 ft. in depth. The foundations were carried down to the marl about 5 ft. below beach level, and owing to the uneven loading of the ground some reinforcing bars are inserted near the base of the wall to prevent the occurrence of shear cracks.

It has been necessary in this chapter to refer at some length to specific examples of coast erosion in order that general conclusions may be drawn which are based rather on experience than on speculation.

It appears that erosion of the coast line may be safely classified, and the classification attempted by the writer in 1914 appears still to be justified in the light of further experience.

That classification is as follows:—

A. Erosion on a natural coast line on which no constructional work has been carried out, which erosion is not attributable to the construction of groynes, sea-walls, or breakwaters elsewhere.

B. As in case *A*, but in which land drainage is a contributory cause to erosion.

C. Damage to artificial protections, such as parades, embankments, revetments, etc.

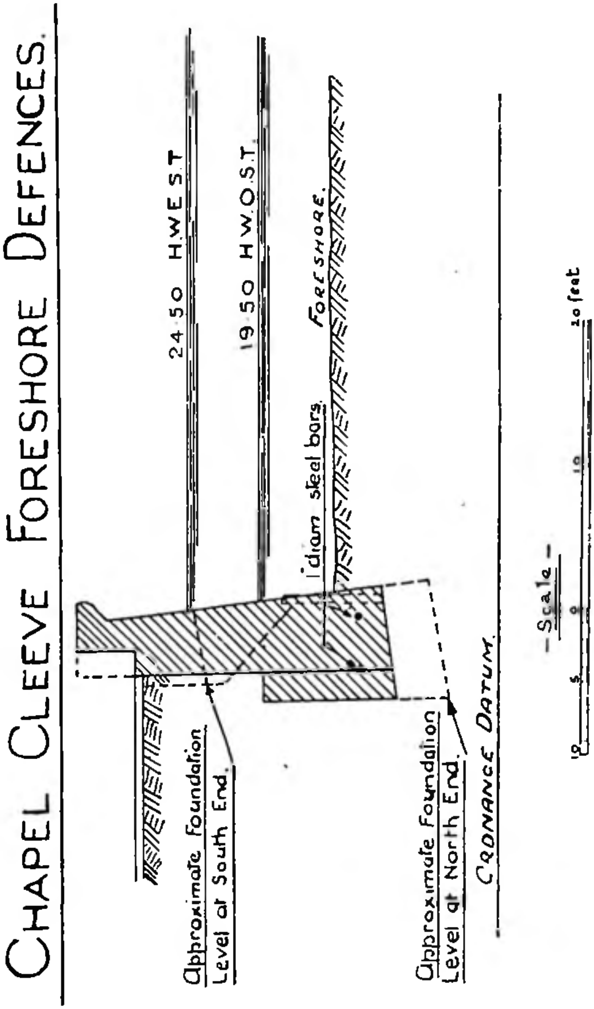


FIG. 61.—Sea-Wall, Chapel Cleeve.

D. As in case *C*, but primarily due to the faulty design of the structures in question, or errors in the lay-out of protective groyning in front of or on the flanks of the lateral defences.

E. Local encroachment on natural land faces due to faulty groyning.

F. By the disintegration and collapse of cliffs consisting of hard rock formations.

G. By the breaching of earthen embankments (or other defences) protecting low-lying land.

It is of importance to the civil engineer to be able to classify the case he has to deal with, as the category into which it falls has an important bearing on the design of coast protection works.

In respect of methods employed to prevent erosion of the coast line and involving the employment of groynes, the following notes may be of service.

It is not proposed to deal with the cause of littoral drift or to discuss the questions of erosion and accretion of beach material in any general sense.

The assumption is therefore made that in any given locality there exists the phenomenon known as "beach travel," evidenced in the lateral movement either of mud, sand, or shingle. On a tidal coast line this assumption favours the rule rather than the exception. Further, the term "beach travel" must be understood to be the balance in favour of travel in one direction over travel in the opposite direction, *e.g.*, observation on a given area over a long period might show x tons displacement of beach materials Eastward and a lesser estimate of y tons displacement Westward. The result would be the positive factor $x—y$ tons represented by an insistent beach travel Eastward. The lesser term y must be by no means ignored, since during the periods of "minor movement" the functions of coast defence works are reversed and marine works frequently must be regarded for such periods as works actually detrimental to the conservancy of the foreshore which they are designed in the end to maintain.

The protection of lands above high water may be effected by the construction of sea-walls—a costly expedient, but sometimes a necessary one. Such undertakings (excluding pleasure promenades) would clearly not be required if the existing profile of a foreshore could be maintained or raised, and it is with this problem that it is now proposed to deal.

The magic word "groyning" is supposed to represent a panacea for all evils. In the first place, the term is misunderstood, and in the second place, it is not a universal cure.

Dealing first with the construction of a groyne on a foreshore previously suffering from erosion, Fig. 62 gives a typical section of groyne as commonly met with on our coasts, and it represents about as unscientific a design as could be evolved. The arrows in the diagrams represent the assumed direction of beach travel in each case. After such a groyne is constructed in this country, and prior to the Autumn gales, a cross section through the groyne may safely be presumed to follow a beach level shown diagrammatically in Fig. 63.

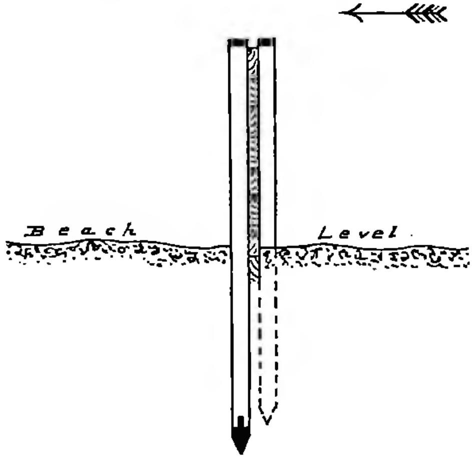


FIG. 62.—Typical Section of Groyne.

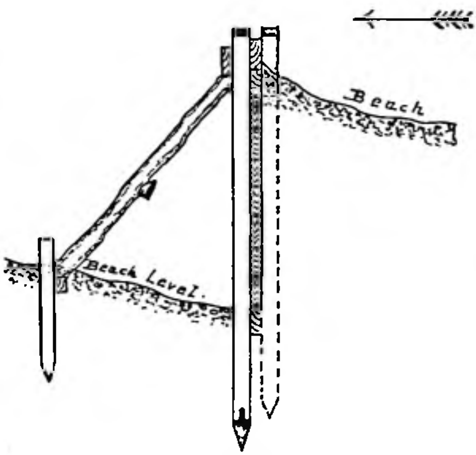


FIG. 63.—Beach Accumulation against Groyne.

The shingle banks up on the weather side and becomes eroded on the lee side, when measures have to be taken to strut up the groyne to resist the heavy lateral pressure exerted by the accumulation

of shingle on one side, and planks have to be inserted at low levels to compensate for a lowered beach level on the other. Much may be done to improve this type of groyne by omitting the upper planks to start with, and adding them as the beach level rises, but at the start there will always be erosion on the lee side, and it may temporarily be of a serious character. As a type of groyne for constructing at the extreme limit of a foreshore to arrest drift material, it may be useful if gradually raised, and the

FIG. 64.—Adjustable Timber Groyne.

photograph (Fig. 64) illustrates such an adjustable groyne recently constructed and performing useful service, as may be seen from the accretion of beach material on the weather side.

If these groynes, as above described, are not of the adjustable type, during the reversal of prevalent conditions, when the direction of beach travel is altered, the evils of erosion may well be accentuated.

The foregoing physical effects are not restricted to timber groynes, as they apply in general to any solid obstructions

of the breakwater type projecting into the sea which (with the exception of certain reinforced concrete groynes of special design) suffer from the additional handicap of not being adjustable.

The maritime engineer has, therefore, been led step by step to the necessity of evolving a design based on the assumed existence of beach travel in the first instance, and having for its object the arrest of beach material along a defined strip of coast line, the ideal result being to collect a foreshore at a natural angle of repose immediately above high-water mark while projecting artificial works seawards as little as possible.

The disadvantages of the old-fashioned groyne are many, and may be summarised as follows:—

1. They are costly, owing to the general necessity of their being carried right out to low-water mark.

2. They obstruct the foreshore and are sometimes dangerous to small craft.

3. They are liable to destruction in heavy weather.

4. They cause local erosion at the "root" or shore end of the groyne.

5. If they are adjustable, they require constant attention.

6. Additional strutting work is usually called for owing to differences in beach levels on either side.

It is only fair to quote the advantages of such groynes. These are:—

1. On the whole, if projected at suitable angles to the shore line, a balance is usually caused in favour of accretion against erosion.

2. They do not require the services of an expert to design.

One is tempted to add the fact that they form an unending source of delight to young children at pleasure resorts, as presenting possibilities of hazardous climbing operations, though perhaps the cost of the groynes does not equally delight the ratepayer.

The problem, therefore, resolves itself into a design of works which will (*a*) catch and retain the bulk of foreign material reaching the area in the direction of prevalent travel;

(*b*) prevent the escape of accumulated material when this direction of travel is reversed.

It would seem that something in the nature of a "non-return" weir is really required to deal with the travel of beach material with a crest which would represent the level of the foreshore it is desired to attain.

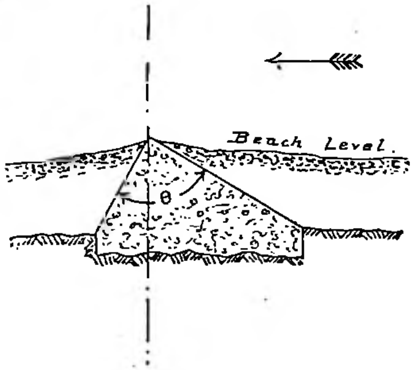


FIG. 65.—Non-Return Groyne.

If Fig. 65 is regarded as purely diagrammatic, it affords some interesting food for thought. This theoretical groyne is essentially short, being intended for construction only in the vicinity of high-water mark. The arrow again indicates the direction of predominant beach travel as carefully verified by previous survey and observation. If we can imagine the apex of this groyne to have an adjustable angle "θ," it is clear that, by experiment in any locality, an angle can be arrived at over which the shingle can be driven by gales of a predominant direction, but which precludes, to a large extent, any back travel when weather conditions are reversed.

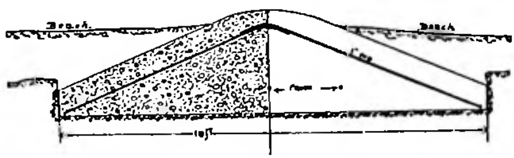


FIG. 66.—"Weir" Groyne.

Groynes of such design would probably be found to be quite effective where the predominant direction of travel is very marked. Where, however, the balance is very finely held, it follows that the weather and lee slopes become almost equal angles and the groyne functions only owing to the usually lesser combination of the weather conditions causing minor travel. The theoretical design also becomes much

modified in practice. Nevertheless, the author has found that the principles outlined above are corroborated by actual experience. During the construction of certain groyne and stockade works in the autumn of 1921 on the West Coast, the foundations of a heavy sea-wall were threatened by the undermining action of the sea due to the erosion of a shingle and shale foreshore.

A series of short concrete groynes, only about 25 ft. long and of section shown in Fig. 66, were constructed and stood

FIG. 67.—"Stub" Groynes.

well above the foreshore, but in a couple of months they were almost submerged in shingle, as illustrated in the photograph (Fig. 67).

It is noticeable here that the crests of the groynes are only just visible. The function of these groynes is undoubtedly assisted by the lateral stockades, but the groynes were nevertheless responsible for securing the safety of the old sea-wall and the new works by virtue of the marked "beach building" qualities exhibited under difficult conditions and with a very slight predominance in direction of beach travel.

Such groynes seem to possess the following material advantages:—

1. They are short, and, though of expensive section, should generally be cheaper than much longer groynes of vertical section.

2. They are not unsightly and soon get buried in sand or shingle.

3. They are not dangerous to small craft inshore.

4. There is no great difference in beach level between the lee and weather sides of the groyne.

5. They can be more quickly constructed than longer groynes of the vertical type.

6. Results should be obtained quickly.

In the specific case illustrated in Figs. 66 and 67 these concrete "stub groynes," as they were termed, presented no foundation difficulties, being constructed on soft shale foundation. There appears, however, to be no insuperable difficulty in bad ground in constructing such works in timber or light reinforced concrete, when a hollow section would, of course, be preferable.

CHAPTER VII

STRUCTURAL PROBLEMS ON NAVIGABLE RIVERS[1]

Responsibilities of River Conservators -- Effects of Reclamation on Navigation—Effect of Barrages—Relative Advantages of Training Works and Dredging—The Conservancy of Adjacent Lands—Critical Points of Liability to Flooding — Discharge of Land Drainage —General Regulations Governing Construction of New Quay Frontages and Jetties.

THE efficient maintenance of navigable rivers serving industrial areas involves a close study of several specialised subjects. The Conservators of such rivers are responsible for all riparian interests as a whole, and do not exercise their authority necessarily in assisting the development of river-side premises owned by any one industrial undertaking. This fact is too often overlooked, and leads often to wasted effort and expense. It is no light task to maintain a just balance between all parties so that each may share the common waterway with equal advantage.

Apart from the question of sewage outfalls, which is not dealt with in this chapter, the subjects involved, given generally in their order of importance, are as follows :—

(1) Navigation.

(2) The conservancy of adjacent lands, *i.e.*, prevention of flooding, etc.

(3) Discharge of land drainage.

(4) Construction of new quay frontages and jetties.

These points are dealt with below in sequence.

Navigation.—Where great natural facilities exist with deep water over a rock bed the problem of maintaining

[1] Reprinted from an article appearing in the *Contract Journal* under dates 22nd February and 1st March 1922.

navigation obviously presents few difficulties. In other and more frequent cases, however, the problem of maintaining navigation will always become more acute as the river-side lands develop and the river banks become more and more "canalised." At first sight this seems a curious anomaly, but the reason is not far to seek. Before embankments, wharves, jetties, bridges, etc., come to be constructed, a river in its natural state has a larger tidal compartment, and the effluent, being larger in volume, the lower channels tend to maintain themselves by natural scour. A river in this condition has usually deep water in its lower reaches, though the amount of detritus brought down by flood water and subsequently deposited at the mouth of the river by the ebb tide is large, and usually results in the formation of bars and shoals in the estuary. With the gradual process of land reclamation and "canalising" of the river banks the tidal effluent is reduced in volume and the flooded lands reduced in area. The tendency to form shoals and bars becomes less marked as regards the volume of material to be deposited, but the scouring effect in the fairway is lessened. Such shoals and bars as remain then tend to move inwards as the "point of deposition" changes, and navigation becomes increasingly difficult until artificial means are resorted to, such as the dredging of entrance channels or the construction of training works. Dredging in the lower reaches affects the tidal range in the upper channels, tending to increase it (see Cartwright Reid on the "Relative Advantages of Dredging and Training Walls," Inst.C.E., 1921), whereas the construction of training works to maintain the entrance channel decreases the tidal range in the upper reaches.

As regards navigation in the higher reaches of rivers serving industrial areas in this country, many schemes for "dockising" the river above a certain point have been projected. Such schemes are admittedly expensive and introduce many contingent difficulties, one of which is the fact that the construction of a barrage across a tidal river involves the necessity of providing a system of locks, and these must have a traffic capacity equal to that existing at the time of construction, with presumably a margin for the future expansion of trade. On either economical or physical

grounds this requirement generally kills the scheme. Nearly 400 vessels in normal times pass Gravesend, on the River Thames, every twenty-four hours, going either up or down stream. On the assumption that two main locks were provided, this would mean locking ships up or down stream at an average of eight or nine ships an hour. This rate, at certain periods of the day, would perhaps be even doubled or trebled, and the congestion and delay would be enormous, even if two or three vessels amongst the smaller craft could be "locked through" at one time. The advantage, for example, of the Port of London Authority having some miles of quay frontage below London Bridge situate along a dock floatage instead of a river is obvious, but it is impracticable to secure, and the same argument applies to most if not all British rivers.

The author believes that the great advantage of a free tidal waterway serving great industrial centres is now appreciated on all hands, and he is of opinion that no works should be executed of a character which definitely tend to obstruct the free flow of the tide. The construction of a barrage raises at once the following points: (*a*) The question of dealing with sewer outfalls above the barrage; (*b*) the discharge of surface water channels previously maintained by tidal flaps, sluices, etc.; (*c*) the prevention of silting in the dockised portion caused by sediment brought down from the upper river and its tributaries, and also shoaling below the barrage (see R. F. Grantham, M.Inst.C.E., on the "Effect of Sluices and Barrages on the Discharge of Tidal Rivers," 1921); (*d*) the unsuitable character of tidal wharves and jetties for permanent submergence, *i.e.*, question of repairs, etc.; (*e*) continuous instead of intermittent pressure on river banks, *i.e.*, possibility of failure of embankments owing to continuous seepage in one direction into low-lying lands without the compensating drainage.

For the above reasons, therefore, it is apparent that the construction of barrages and locks (if they can be avoided) is against the best interests of navigation, and tidal river improvements appear to be best confined to dredging or the construction of training works with intelligent restrictions in licensing new works undertaken by private wharfingers. As

to whether dredging or training works are the most suitable for maintaining navigation channels, the following extracts from a paper read before the Engineering Conference, Section II., Institution of Civil Engineers, last year, by Mr H. Cartwright Reid, M.Inst.C.E., correctly sum up the problem, as follows:—

"The physical features differ so materially that it is impossible to lay down universal laws in regard to their treatment, and to most general axioms there would always be a number of exceptions.

"The advantages of 'dredging' may be summarised as follows: (*a*) Immediate advantage of increase of depth, and definite results in depth and width; (*b*) can be carried out gradually and experimentally; (*c*) tends to combine the ebb and flow current in the same channel; (*d*) helps to keep the level of the river below the banks; (*e*) can be economically combined with reclamation; (*f*) is economical in first cost.

"The disadvantages are: (*a*) Where strong littoral currents exist maintenance cost may be excessive; (*b*) in many other cases a considerable amount of maintenance is required; (*c*) traffic is obstructed by the dredging plant.

"Training works, on the other hand, have advantages and disadvantages in the converse."

The Conservancy of Adjacent Lands.—As industrial development takes place along a river bank the question of maintaining the river-side lands becomes important.

In the first instance, the gradual canalising of the river causes an increased amplitude of tide, and the marsh lands which usually exist on the lower reaches, hitherto of small value and more or less derelict, are gradually reclaimed, and as the subsoil dries out the surface level sinks. This latter consideration does not, of course, apply to all rivers, but to the majority, since it is seldom that large growing cities with their factories and industries are located on banks of rivers running through rocky gorges, the industrial centres generally being confined to relatively low-lying lands.

In this country such lands have gradually come under the control of bodies of Level Commissioners, who are responsible for maintaining the land drainage outfalls, the dykes clear of obstructions, and for introducing culverts as the ground is

built over, and defending such ground against the possibilities of flooding by abnormal tides in the river. The powers of such Commissioners are either based on ancient statutes or on the Land Drainage Act of 1861, as amended by the 1918 Act.

The boundary line between the jurisdiction of such Commissioners and the main river conservancies is generally an obscure one. Although each public authority relies on statutory powers given by public or private Acts of Parliament, the general idea followed and traceable in each Act is that of separating the functions of the two bodies in administration rather than an attempt to establish a physical boundary line. Whether this precedent is to the advantage or disadvantage of the public it is hard to say, but it appears that the Commissioners have to maintain the land, whilst the river authority has to maintain the river, especially from the points of view of navigation and pollution. It is therefore not open to sewer or level Commissioners to construct works which obstruct navigation, or are likely to cause pollution, without the sanction of the main river conservancy, and likewise the main river conservancy and private wharfingers are restricted from carrying out works affecting embankments, land defences, or sewer outfalls without obtaining the sanction of the Commissioners.

The possibility of flooding adjacent lands is a big problem, as any breach in river embankments or other river defences is extremely difficult to repair, and the damage caused by a single tide may often result in considerable areas of low-lying land being flooded for an indefinite period.

Through storm conditions at sea and in the estuaries of rivers abnormal tides sometimes occur in the higher reaches, and precautions for the defence of low-lying lands have to be taken on observations based over a long period, as an exceptional tide may not recur with the same intensity for a generation. For example, the high tide in the Thames on 1st November 1921 reached a level which had not been experienced for seventeen years. It is perhaps not generally appreciated that there is in every river a critical tidal point, and that it is within a certain zone surrounding this point that flooding may most likely be expected to occur.

Explanation of this is of interest. On an abnormally high tide with river embankments at standard level the tide will overflow first only at one point. This phenomenon is frequently experienced in the case of creeks running up from the river in cases where the embankment remains intact at the junction of the creek with the river and at the head of the creek, but is breached at some intermediate point. In the same way, with the main river itself, there is some intermediate critical point where flooding will first be experienced, varied as to position only by any special precautions taken in raising the embankments.

It should be understood that these remarks apply to conditions where the river embankments are maintained throughout in good condition, and do not apply to flooding caused by an embankment or wharf coping being below standard level, which is, of course, a special and frequent source of danger known to all engineers.

As regards the determination of the critical point above referred to, Fig. 68 may be of interest. The tidal range of a river is that of the open sea at its mouth. This range increases as one passes up river and reaches a maximum value at a critical point, then steadily decreasing to zero at the "highest point reached by the tide."

Taking as abscissa the mean tide level, we get a diagram somewhat similar to Fig. 68. It is clear that the standard level of wharves, dock entrances, and embankments will depend on the upper curve, and theoretically will bear a definite relationship to the level of this curve. The diagram does not indicate actual levels and may appear anomalous at first sight, but it is a simple method of explaining the point under consideration, provided it is regarded as only illustrating the comparison between varying tidal ranges and embankment levels.

Assuming that the standard level of defence is 5 ft. above high-water ordinary tides and an abnormally high tide occurs, the effect of such a tide will be to increase the "range" by a definite percentage at all points, and it will thus be seen that the increment in level is greater at or near the point where the tidal range is greatest, leaving there a less margin of safety than on any other section of the river, unless the

level of the defences is proportionately raised, which is seldom the case. The dotted line indicates diagrammatically the increased amplitude of tide due to storm conditions. It is

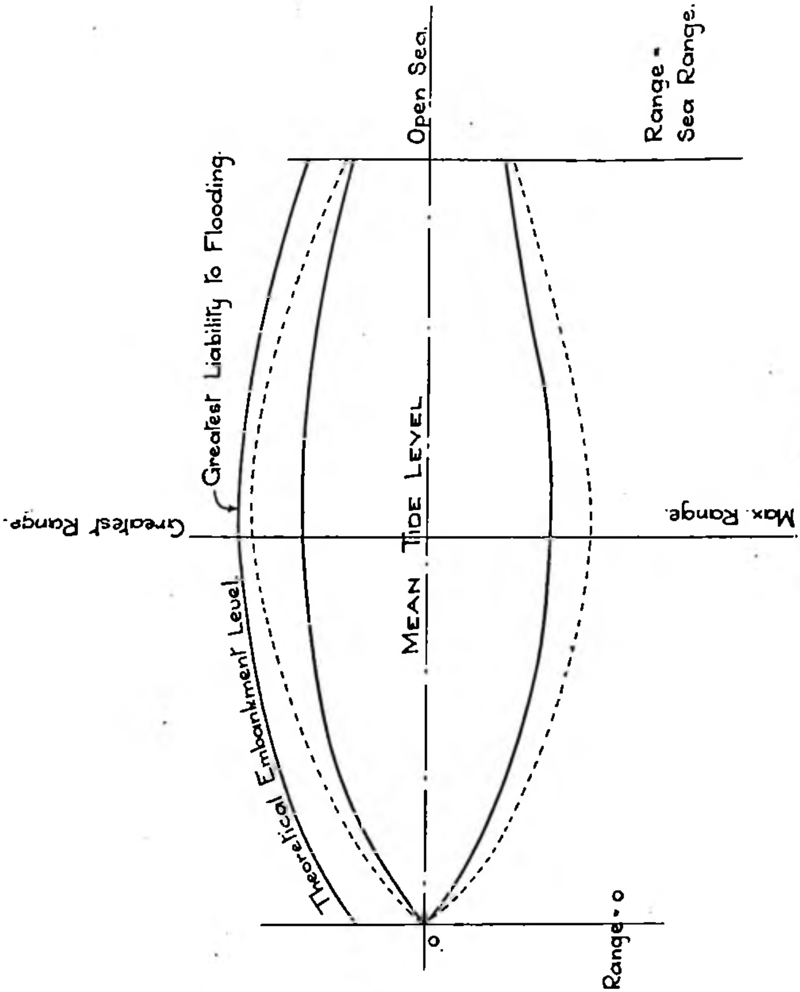


FIG. 68.—Diagram of River Tides.

probable, therefore, that the margin of safety adopted on our waterways is by no means a constant factor, and the lack of a constant margin of safety may well account for many recent floods.

Abnormal tide levels due to storm conditions are further aggravated by the frequently accompanying phenomenon of low barometric pressure which is referred to in a thoughtful communication by Commander E. C. Shankland, R.N.R., F.R.Met.Soc., to the journal *Water and Water Engineering*, under date 20th February 1922.

Discharge of Land Drainage.—The efficient discharge of land drainage is always of great importance, and with the growth of a township, the rainfall, only part of which previously found its way to the river, being partly accounted for by absorption, is discharged rapidly from the roofs of buildings, pavements, yards, etc. The rate and quantity of the "run off," therefore, increases—the rate especially. It follows that many outfalls have to be enlarged as they prove themselves not of sufficient capacity to deal with the new rate of storm discharge.

One common result of the increased freshet from surface water is that old wharf faces of the solid-fill variety become more or less permanently waterlogged, and the backing thus being flooded, the lateral pressure on the wharf face increases and causes bulging in the case of timber-faced wharves, or failure by fracture and movement in masonry or concrete structures.

With the exception of public wharves this aspect of river conservancy is of more interest to private wharfingers than to the actual conservancy board of the river. The interests of the latter are, however, likely to some extent to be affected, as such derelict structures, if not repaired, become a source of danger to shipping and barges.

Another difficulty is caused by the accumulation of mud banks in the neighbourhood of sluices or tidal flaps. It is a well-known fact that a stream from an outfall impinging more or less at right angles on the tidal currents acts to all intents and purposes as a groyne. Such mud banks are commonly visible on the upstream side of such outfalls, the accumulation often rising to a level many feet above that of the invert of the sluice. In the construction of new sluices, therefore, it is advisable wherever possible to lead the effluent culvert on to the foreshore at an acute angle with the river bank so that the discharged water is shot out

in a down stream direction, and not at right angles to the river currents.

The jamming of sluices by wreckage is a common occurrence, and is frequently due to the sluice being constructed with wing walls projecting out into the river. This is a very bad type of design, and wherever possible should be avoided. The face of the outfall should always terminate in a flush wall or the outfall itself be carried out into the open river.

The best type of sluice is where the tidal valves are enclosed in a brick or concrete chamber, the final outlet to the river being free of valves or sluices and protected at the outer end by a close grille.

When sluices become jammed flooding of the land at the back of the wharf or embankment takes place, and the results may be very serious, both in respect of damage to lands and buildings, and as regards the conservancy of the main river. With ground in such a condition the liability of a breach in an embankment is very great.

Construction of New Quay Frontages and Jetties.—It is of considerable importance in the public interest that the policy of any river authority should be fully understood by factory and property owners on the river-side.

In the case of most British rivers with gradual growth of industry round them, there was in the early days no settled policy as to the dimensions or designs of piers, jetties, or wharves erected by private enterprise. As, however, river trade increased and shipping became more dense and navigation more difficult, the obvious danger of granting permits in a haphazard way became apparent, and to-day most river conservancies have a more or less settled policy with which any projected undertaking on the river-side has to conform.

Where the conservators of a river are themselves dock owners or wharfingers, as in the cases of the Rivers Thames and Mersey, it can be readily understood that such bodies are none too anxious to sanction the construction of deep-water quays or wharves at which a general wharfinging trade can be carried on, as there is the possibility of a definite loss of trade and income to the river authority. This point is still

a controversial one, and it is difficult to say whether the present practically unlimited powers of restriction are in general to the public advantage or not. The question is an involved one, and perhaps one on which the engineer or contractor is not really qualified to give an opinion.

There exist on all industrial rivers a number of "single" trades, such as cement manufacture, oil storage, paper pulp importation, coaling staithes, the general engineering trades and others. These "single" trades, apart from general wharfinging, as they grow in extent require statutory authority to construct their wharves, jetties, and quays of suitable characters to carry on their respective businesses. The attitude which river conservancies take towards this class of enterprise is largely governed by the advice of their permanent officials, *e.g.*, that of their engineer as regards the proposed type of construction, and that of the harbour-master in respect of navigation. In the event of a licence to construct being refused there is generally a right of appeal, but it is seldom resorted to.

The following headings indicate the many points which have to be considered by such committees, and it should be appreciated that careful consideration is necessary before decisions can be given.

1. The nature of the trade to be handled.
2. The distance to which skeleton structures such as piers, dolphins, etc., can be carried into the river.
3. The distance to which solid structures such as quays, etc., can be advanced towards the fairway.
4. How existing mooring facilities may be affected.
5. What possibilities there are of "overshadowing" adjacent properties, either by shipping alongside or by the structures themselves.
6. If dredging is necessary, and, in this event, how it may affect the navigation of the river or the stability of adjacent quay walls, factory foundations, etc.

On some rivers the permissible limits for new work are gradually becoming more or less standardised, and although such regulations are not officially issued, they form a precedent on which all applications are considered.

Fig. 69 illustrates in general this limiting policy as adopted by one important river conservancy in this country. From this it will be seen that two limiting lines are defined, more or less parallel with each river bank. One of these lines is known as the "jetty line," and if there are no special objections, this line represents the limit to which piers, dolphins, and jetties of the open or skeleton type can be extended into the river. The second line, or "embankment

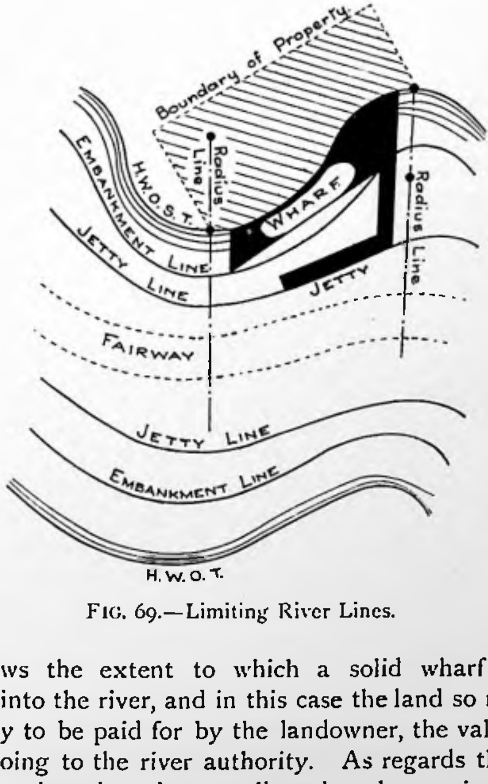


Fig. 69.—Limiting River Lines.

line," shows the extent to which a solid wharf may be projected into the river, and in this case the land so reclaimed has usually to be paid for by the landowner, the value of the freehold going to the river authority. As regards the extent to which projected works are allowed to be carried up and down stream, limits are obtained by "radius lines" which project into the river on either side of the applicant's property. These lines are obtained by connecting the point where high-water mark passes the limit of the property to the instantaneous centre of curvature of the river bank, and projecting

(if necessary) this line into the river. Permanent structures are generally permitted within the jetty and embankment lines and within a specific distance of the radius lines. If the property is small, or the projected works so arranged that shipping would seriously overshadow existing and adjacent industrial frontages, the usual limits may be very much restricted, but where the property is, in the first instance, industrially isolated and clear of existing river moorings, the above limits are usually adhered to.

As all structures projected into a river cause both scour and siltage at different points, it has become necessary to limit the minimum span of open-piled structures, and most conservators, for this reason, favour the "cylinder" type of construction where wide gaps between the cylinders offer little interference to the river currents.

CHAPTER VIII

SCOUR[1]

Definitions of Scour—Differences between Pure Scour and Scouring Effect—Critical Velocities—Scour in Rivers—Effect of Flood and Ebb Tide—Training Works—Value of Surface Observations as to Level of Water—Scouring Velocities.

THE study of scour and scouring velocities is a matter of very great importance to the civil engineer dealing with river and coast problems.

It is first of all essential to attempt some definition of the word "scour." At first sight such a necessity appears an unusual refinement especially to engineers who are not dealing with this class of problem every day, and are liable to consider that the term "scour" covers definite physical phenomena with which they believe themselves to be well acquainted. This is true to only a limited extent. In this way the term has come to be loosely used to cover river or sea-action caused by a variety of factors, and not necessarily by scour alone.

What is commonly called "scour" often comprises wave action or denudation due to definite hydrostatic heads. Instances of this frequently occur on the coast line, where erosion takes place either at the bottom of cliffs composed of clay or loam formations, or in the case of the erosion of shingle, sand, or mud banks. In the former case "wind-wave" action and weathering of the cliff face are auxiliary factors, and scour may be absent or only partially responsible for erosion. In the second case the effect may be caused by "wind-waves" alone, or, alternatively, by scour alone, or a combination of both.

[1] Reprinted from an article in *Water and Water Engineering*, dated 20th March 1922.

The formation of dangerous channels at the toes of sea-wall aprons is again frequently not due to scour alone, but to the definite disturbance of the beach material by the recoil of the wave which, descending, acts very much on the lines of an eroding jet, and this again is a purely hydrostatic action. There are, however, many instances of scour pure and simple, and the writer believes that certain investigations now being undertaken by American engineers, especially in regard to the scour and siltage of the Yellow River in China, will probably throw a good deal of light on the subject.

For the purpose of this article, however, "scour" may be defined as follows:—

"Erosion from the sea or river bed of sand, mud, or other materials, and their deposition elsewhere, such erosion being caused solely by the velocity of the ground currents."

The above is a true definition of scour, and whereas *scouring action* may give results similar to those obtained under the effect of pure scour, such removal and deposition of material is not necessarily caused by scour alone, the additional action of "wind-waves," weather, etc., being collectively or separately concerned.

In cases of pure scour, the greatest movement generally occurs in the case of materials consisting of small particles such as mud or sand. Apparently, at some critical velocity, the sand or mud particles come into suspension in the water, and remain in suspension so long as this critical velocity is maintained or exceeded. This is a very curious action, as it is distinctly different to the popular idea that the particles of matter are invariably rolled along the bottom of the sea or river bed. In other words, if an actual physical experiment is carried out in glass tanks it will be found that the water remains clear under a current of increasing velocity until the critical velocity is obtained, when the particles come into suspension. This action was described recently at the Institution of Civil Engineers by Mr J. R. Freeman, Am.S.C.E. (see *Engineering*, 15th July 1921, p. 121), as being "colloidal," and the critical velocity of sediment passing a 200 by 200 to the inch mesh sieve was determined at 5 ft. per second, or between three and four miles per hour. In

the case of Thames ballast, the writer has found that scour will commence at ground current velocities of about two miles per hour. It is possible, therefore, that with fine materials the actual rolling along of the particles over the sea or river bed plays only a very minor part in the phenomenon of scour.

When pure scour takes place, the material taken into suspension is deposited over some other area, and in designing marine works it is as important to look for and locate this area of accretion as it is to determine the extent of the loss of material at the critical point of scour, for in nearly every case of an obstruction projecting into a river or from a coast line, scour will take place, although, on an open coast line, the effect is sometimes masked by "wind-wave" action.

It is, therefore, first to the scour of rivers that attention should be directed, as in many cases this is pure scour, and if this part of the problem is thoroughly understood, the effect of scour in combination with other action can be more readily understood on a coast line in connection with harbour or coast defence work.

In the case of river quays or jetties, the berthage is frequently approximately parallel with the river bank. The jetty itself causes local restriction of current between the piles, cylinders, or other supports, and to some degree gorges the stream between the jetty head and the shore. There is, therefore, locally a sharp and marked increase in the velocity of the stream. One of the best examples of this was experienced at Purfleet in 1909, when the increase in the stream velocity caused by a jetty was so great that it actually caused a partial collapse of the adjacent river embankment. This ground-scouring action is shown diagrammatically in Fig. 70.

It is fairly obvious that for a defined length of river having a constant hydraulic cross section there is a certain average velocity at any moment which must be maintained to evacuate the tidal and fresh water discharge, and alternatively to take the influx of the flood. As this velocity is sharply increased as the stream of a river passes an obstruction, it follows that a point must be reached (following

the direction of the current) where the velocity will fall below the average of that section of the river so as to maintain the average velocity taken as a whole. It is at this point where deposition or siltage will take place. It follows that a very careful study of sub-surface velocities should be made where any question of scour or siltage is concerned, and it must be borne in mind that very slight changes of velocity caused by the construction of river works will entirely alter both the site of initial scour and the site of resultant deposition.

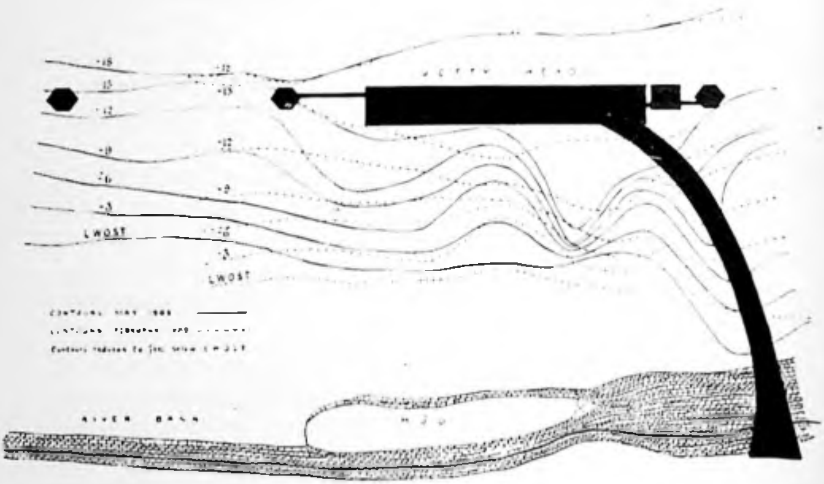


FIG. 70.—Progressive Underwater Scour at Purfleet, Essex, 1908-09.

In tidal waters it is necessary to determine which is the scouring tide (*i.e.*, the flood or the ebb), and, as an illustration of scour caused by an ebb tide, the case of a deep-water quay at Greenhithe may be quoted. In this case the original quay was 300 ft. long, and the ebb tide was the scouring current, which is frequently the case on the River Thames. The scour was the more marked owing to the works being built on a concave bend of the river. Scour began at the upstream dolphin and continued along the face of the quay, which was 300 ft. long, the area of deposition commencing apparently about 150 ft. beyond the downstream end of the quay. It was decided in 1920 to extend this quay head

upstream by 300 ft., thereby making the quay face 600 ft. long in all. The writer pointed out to the wharfingers that dredging would probably have to be resorted to from time to time at the downstream end, as there would be a decided tendency to bring the area of deposition upstream, and that this might tend to "blanket" the lower end of the berth at the downstream end of the quay face. This, in fact, was what occurred, and, on making inquiries some six months after the new works had been finished, it was ascertained that river mud to the depth of 6 ft. had been deposited at the lower end of the quay. The physical result in this particular case was not of very great importance, but under other conditions of ownership, etc., it is conceivable that a great deal of annoyance and possible damage might have occurred.

A close study of scour where soft material is concerned will very often enable channels to be deepened and berths improved by training works of a temporary character, which in some cases are a cheaper expedient than dredging, with the additional advantage of giving a permanent improvement in the depth of water and eliminating dredging, which is, of course, a constant source of expense.

In dredging fine sand a special class of dredger is often required, as the destruction of the dredger buckets and gear of ordinary ladder dredgers when working through fine sand is serious. An example of this was the wear and tear caused to bucket dredgers unexpectedly meeting fine sand deposits at Shellhaven, Essex, in 1921. This objection vanishes if a natural scour can be secured.

It is clear that it is seldom, if ever, possible to get photographs showing the effects of scour *per se*, for the simple reason that this invariably takes place below the lowest water level where no wind and little, if any, wave action can take place. Where scour, weathering, and wave action take place together, however, very descriptive photographs can be obtained, and Fig. 71 shows such an effect at Southwold. This photograph was taken in 1906.

It must be remembered that effects almost as bad as that illustrated do occur below low-water level, and may be caused by scour alone. In rivers of narrow width, where

wave action is reduced to a minimum, scour of almost a pure character may take place between high and low water mark. One instance of this occurred at Chatham, on the River Medway, in 1913. This scour occurred (as is usually the case) on a concave bank of the river, which consisted of clay and artificial debris. This action, which was affecting Government property (Gun Wharf), was ultimately arrested by work carried out to the design of the author's firm. The design comprised an articulated reinforced concrete revet-

FIG. 71.—Scouring Action at Southwold, Suffolk, 1906.

ment, and the work was carried out as illustrated in Fig. 27, p. 44. The toe of the revetment was secured by means of timber piling.

Scouring effect caused by obstructions at right angles to the current has been previously referred to, and it is a common sight to see definite "weir action" taking place where the current is gorged by such obstructions. In such cases it is essential to obtain accurately the difference in water levels above and below the obstruction under the worst conditions of current. This is admittedly a surface observation only,

but simple hydraulic formulæ show that a difference of 3 in. in level must cause a local increment in velocity of 4 ft. per second, and in this respect it is useful to quote some old data given by Professor Rankine in his standard work on "Civil Engineering." These are given below, with the velocities also reduced to miles per hour:—

TABLE X.—SCOURING VELOCITIES

	Feet per Second.	Miles per Hour.
Soft clay	0.25	0.17
Fine sand	0.50	0.34
Coarse sand, and gravel as large as peas	0.70	0.48
Gravel as large as French beans	1.00	0.68
Gravel, 1 in. in diameter	2.25	1.54
Pebbles, 1½ in. in diameter	3.33	2.27
Heavy shingle	4.00	2.73
Soft rock, brick, earthenware	4.50	3.08
Rock, various kinds	6.00[1]	...

[1] And upwards.

The writer has found, in his experience, that the figures given by the late Professor Rankine are amply supported by experience to-day, and to summarise it appears that most ordinary materials are "scourable" at velocities of under three miles per hour, and soft materials at velocities of even less than a quarter of a mile per hour. This is a fact which appears to be very imperfectly realised, and when we come to consider that a local 3 in. difference in water level (by no means uncommon) will produce a current of nearly three miles an hour on the surface, it becomes apparent that this very marked local increase must be transmitted in some degree to the ground currents, causing scour of the character experienced at Purfleet, and referred to above.

When the civil engineer, therefore, is called upon to deal with a case of scour, he should first pay very careful attention to any local differences in the surface levels of the stream, which will give a definite indication as to the scouring velocities occurring below, and are a better indication than observations by surface floats, which take an appreciable

period to reach the stream velocity and are immediately checked on reaching open water.

It is also important to take sub-surface velocities either by means of current meters or pressure tubes, finally obtaining the direction of the sub-surface velocities by means of submerged floats. It cannot be too strongly emphasised that this class of observation work, which is expensive, is, however, essential in order to determine the cause and site of detrimental scour or to enable the design of training works, wharves, and coast defence works to be undertaken with satisfactory results.

CHAPTER IX

DEEP-WATER QUAYS[1]

Deep-Water Quays compared with Dock Floatages—Reinforced Concrete and Timber Structures—Effects of Collision—Costs—Mooring Facilities—Foundations—Progress Diagrams.

THERE has always been some attraction to engineers in constructing deep-water berths in partially protected tidal waters, as against the more costly equivalent accommodation provided by dock construction. There is no question that for the discharge and loading of certain cargoes, dock accommodation is essential with the relative levels of ship and quayside unaffected by the rise and fall of tide. The heavy cost of deep concrete or masonry quay walls, locks, gates, and other accessories, however, brings up the cost per lineal foot of quayside greatly beyond that for which, under favourable conditions, deep-water tidal jetties or quays can be constructed. The author's actual experience has been principally in the Thames estuary, where his firm have been either responsible for construction, or had to report on many structures of this type. The River Thames and its estuary offer at points suitable sites for this class of berthage, and from Erith down river there has been for a number of years steady industrial growth along the river-side, especially amongst trades seeking deep-water accommodation and for whose special class of traffic tidal berths are suitable.

By the end of the Great War ships up to 20,000 tons deadweight were berthing in this manner in the Thames estuary. The trades for which this relatively cheap class of quayside accommodation appears to be suitable are coal, oil, paper, and cement, or, in short, those trades which load or discharge principally in bulk, barrels, kegs, or sacks, and do

[1] Reprinted from an article in *Engineering*, dated 31st March 1922.

not necessarily require transit shed accommodation within a few feet of the berthage. The policy of the Port of London Authority, while naturally tending to the protection and maintenance of dock traffic, has of recent years been liberal towards private enterprise down river, thereby giving industrial undertakings every encouragement to construct their own quayside works, and the author has never met with anything but encouragement and assistance from the technical officers of the port.

The cost and speed of construction vary largely with the design employed and the materials selected for the works. Generally there is both need for economy and rapidity of construction since, for some obscure reason, those responsible for establishing river-side factories or warehouses postpone decisions as to their marine works till the last moment, not clearly foreseeing that this section of their enterprise is, for reasons obvious to the maritime engineer, the slowest section of the constructional work.

The principal types of jetties and wharves in the Thames are sharply divided into two classes, each permitting of many variations in design, viz.: (*a*) Reinforced concrete; (*b*) timber.

With reinforced concrete, as with every departure from older practice, it is hard to determine to whom should be given the actual credit for the innovation. Reinforced concrete was originally developed as a novel form of construction by French engineers, and had reached a great vogue also in this country prior to the Great War, numerous specialist "licensing companies" having sprung into being. The author, amongst others, was at first very favourably inclined towards this method of construction, as applied to this particular branch of engineering, but experience has now taught him where fully to appreciate its utility and economy, and where to realise its limitations.

He has formed the conclusion that, under post-war conditions, it is not always the most suitable type of construction to employ for deep-water tidal berths. This will be regarded as a sweeping assertion, and it is open to much criticism by members of our profession who are devoted adherents to the school of *béton armée.* It is there-

fore proposed in this chapter first to advance arguments, and then to support them as far as possible with such facts as are available.

It is first necessary to limit consideration of this problem to definite geographical limits—since, as applied elsewhere, the arguments here advanced would admittedly be unsound and misleading. The arguments are therefore confined to the coast, rivers, and estuaries of the United Kingdom, with their varying tidal ranges of from 6 ft. to nearly 60 ft.

Tidal berths generally become impracticable in waters having a range at ordinary tides of much more than 25 ft. The rivers, coasts, and estuaries within this tidal range are subject generally to variable weather, fog, and tidal currents, and the structures forming tidal berths are therefore liable to heavy impact, beyond that experienced in docks, from shipping when berthing, and they are further liable to collision. In a busy river and estuary, such as the Thames below London Bridge, this is especially the case. In general, most deep-water quay designs comprise piling as an essential part of the structure. The actual magnitude of the lateral forces exerted on a jetty by impact from ships making the berth or colliding with it is, of course, indeterminate, but it is obviously less the greater the safe "give" or "spring" of the structure. Reinforced concrete piles, cylinders, or structures do not adapt themselves to movements of this character without disintegration. Further, in the case of actual collision, where extensive damage is often caused in the case of reinforced concrete structures, it is difficult to trace the extreme limits of damage or to say how far the strength of the structure has really been impaired. That these are not prejudiced objections is proved by the illustrations here reproduced.

The photograph, Fig. 72, shows the result of a collision between one of the liners of the Messageries Maritimes and a deep-water jetty built in reinforced concrete. The vessel in this case was about 6,000 tons register. There were 30 ft. of water at low tide at the jetty in question, and the ship was off her course in a fog. The collision was of the "stem-on" type, and the damage shows clearly in the illustration. It

is difficult for any engineer to say to what extent such damaged work must be cut out and replaced. The work of cutting out reinforced concrete is hard and costly, while special plant in the form of shuttering, concrete mixers, etc., are required. The work of reparation takes a long time, and

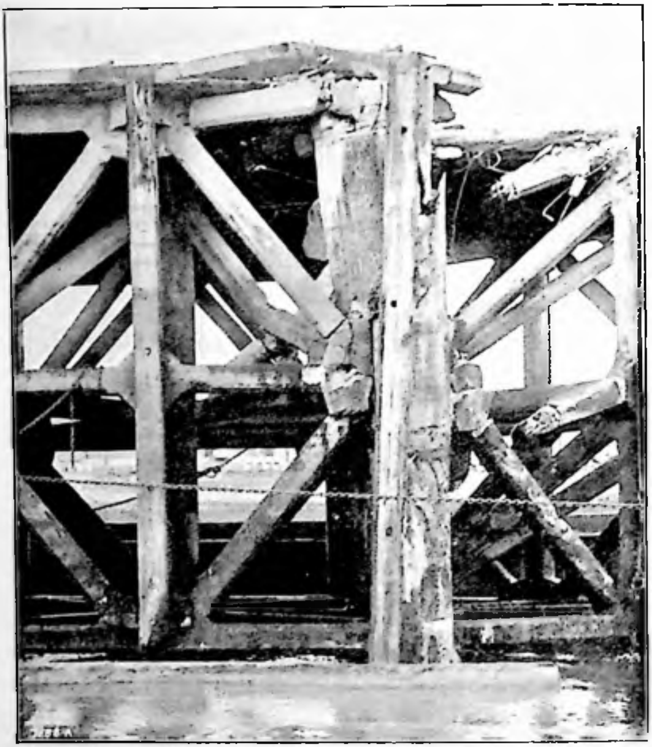


FIG. 72.—Reinforced Concrete Quay, damaged by 6,000 ton Steamer.

in consequence the period over which the berthage cannot be used is considerable.

The photograph, Fig. 73, shows the result of a collision between a ship of 11,000 tons register and a timber jetty used for the same class of trade. The accident in this case was again due to a fog. The extent of the damage is clear and

defined. The damaged work can rapidly be dismantled. No special plant is required for reconstruction, and the work of reparation being rapid, the period during which the jetty cannot be used is short.

As regards cost and rapidity of construction, the author's

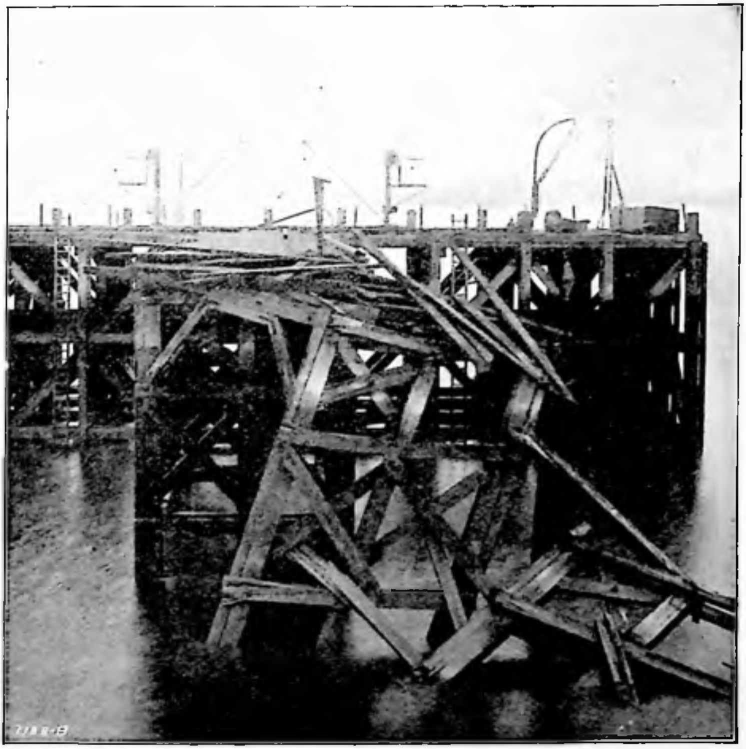


FIG. 73.—Timber Quay, damaged by 11,000 ton Steamer.

firm, as consultants, were responsible for two extensive deep-water berths constructed simultaneously in the Thames estuary. Both works were commenced in the autumn of 1919, and were amongst the first marine works carried out by private enterprise following the conclusion of hostilities.

One undertaking (Fig. 74) was situated on the Essex shore on the north bank of the river, and was carried out in reinforced concrete (Coignet system). It consisted of the extension of a deep-water berthage for an oil storage depot. The tidal range was about 18 ft. with 29 ft. of water at low-water ordinary spring tides. The length of the new approach was 560 ft., with a width of 18 ft., the length of new deep-water berthage being 420 ft., with a deck width of 40 ft. throughout. The total deck area of the new works was approximately 31,100 sq. ft. The contract was let in August 1920, and the works were estimated at £80,000, or £2. 11s. 5d. per sq. ft. of new wharf accommodation obtained. Figs. 75 and 76 illustrate the duplex travelling pile-driving frame used by Messrs Mowlem & Co. on this contract.

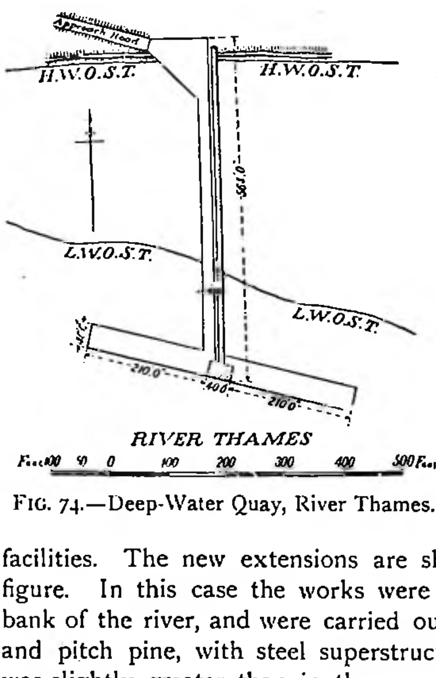


Fig. 74.—Deep-Water Quay, River Thames.

The second undertaking (Fig. 77) was very similar in character, representing as it did the extension of existing wharf facilities. The new extensions are shown unshaded in the figure. In this case the works were situated on the south bank of the river, and were carried out in creosoted Oregon and pitch pine, with steel superstructure. The tidal range was slightly greater than in the preceding case, but with a depth of only about 17 ft. at low-water ordinary spring tides. Owing, however, to the bad nature of the ground below the river bed, the length of piles required was approximately the same. The length of the new section of the approach was 455 ft., with a minimum width of 8 ft. The length of the new deep-water berthage was 303 ft., with a width of 52 ft. throughout, and three splays were provided, as shown in the

plan. The total deck area of the new works was approximately 24,600 sq. ft. The contract was let on 26th July 1919,

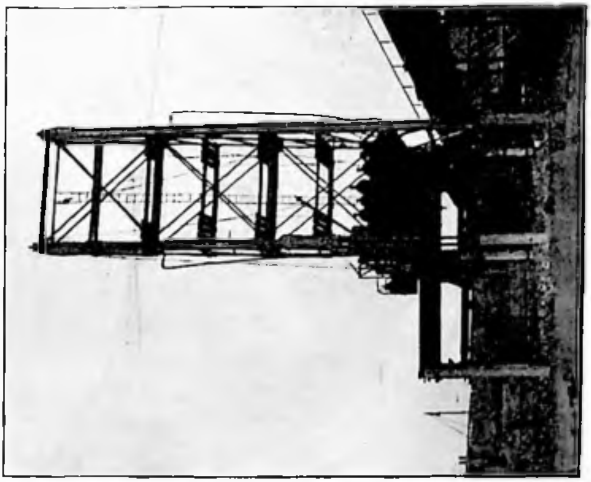


FIG. 76.—Duplex Pile-Driving Frame on Scar End of Works, Shellhaven.

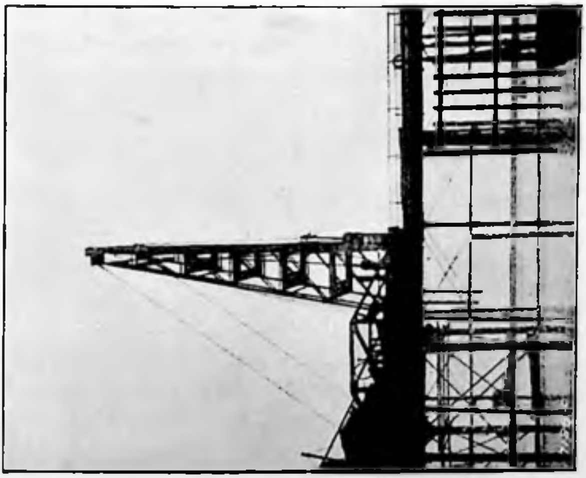


FIG. 75.—Duplex Pile-Driving Frame.

and the works cost £70,783, or £2. 17s. 8d. per sq. ft. of new wharf accommodation obtained. The Main Quay extension is illustrated in Fig. 78.

At first sight these comparisons appear to favour the use of reinforced concrete. The timber jetty, however, was completed on 31st March 1921, at which date the reinforced concrete jetty was less than half completed, with some evidence that the original estimate was likely to be exceeded.

There remains, of course, the question of maintenance. It has been suggested that something in the nature of 10 per cent. per annum is required to maintain a timber jetty in the River Thames. This, in the author's opinion, is excessive. If

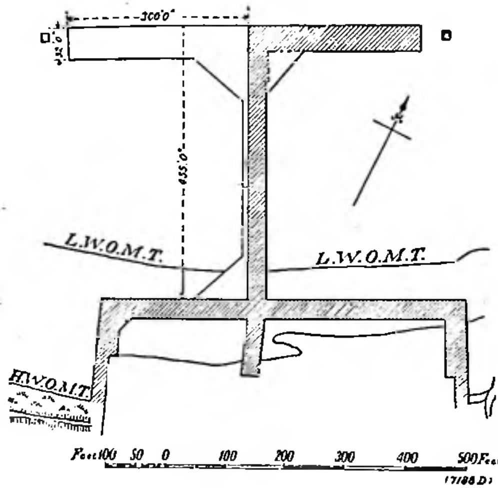


FIG. 77.—Timber Deep-Water Quay.

neither the reinforced concrete nor timber quay were used for their intended purposes, and they were immune from collision, the reinforced concrete structure would naturally be the more enduring. Subject, however, to collision and damage by shipping as such quays are, it is fairly obvious that the more economical proposition for the practical wharfinger is still the timber quay. These have acquired a bad name solely through the almost criminal negligence of their owners in allowing them to fall into decay.

An interesting example of reinforced concrete work exists in the new deep-water quay now under construction at Thames

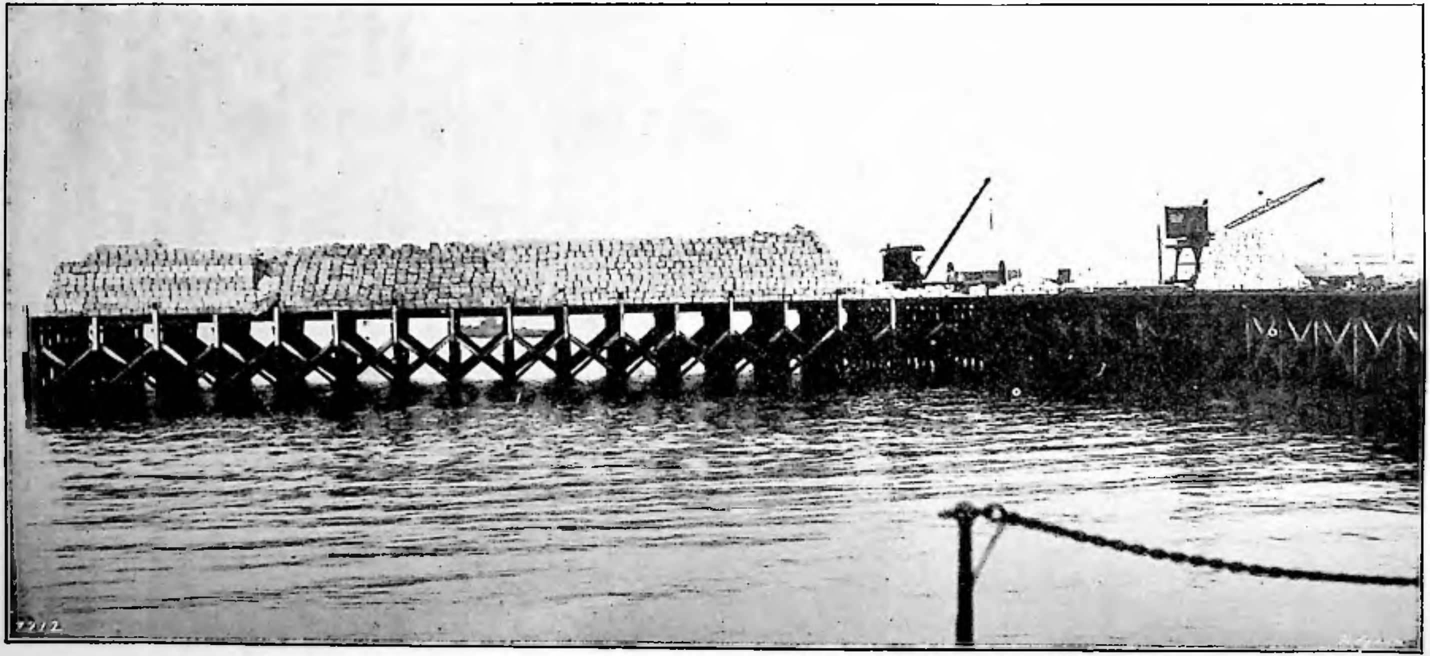


FIG. 78.—Extension of Empire Paper Mill's Timber Deep-Water Quay, Greenhithe, 1921.

Haven for the London and Thames Haven Oil Wharves Ltd. Fig. 79 gives an idea of the type of construction shown in perspective. The general outline of the design was laid down by the author's firm, and the Considere Construction Co., of 5 Victoria Street, S.W. 1, designed the reinforcement.

The cylinder type of construction has been selected, the cylinders being spaced at 30 ft. centres. The bracings, struts, and walings have been designed so as to enable them to be pre-cast, and these works, it is hoped, can be regarded as representing good modern practice in reinforced concrete for wharf accommodation in a tidal estuary. It will be noted that some saving in initial cost may perhaps be effected in this wharf by the reduction of deck area, the cylinders in the rear of the structure being carried to only a few feet above low water with rising struts to deck level on the quayside. The structure has been so designed as to permit of the full deck width being adopted if required at a later date, by continuing the superstructure in rear of the short access deck now provided for along the quayside. The quay has been designed to deal with the oil trade, and for this purpose the only deck required is really for the purpose of access to the ship, and for accommodating the pipe lines and handling the mooring ropes. The prime function of this berthage is to enable ships to come alongside in the open river and to be rigidly held there either while discharging or receiving oil cargoes. A minimum depth of 30 ft. at low water will be secured.

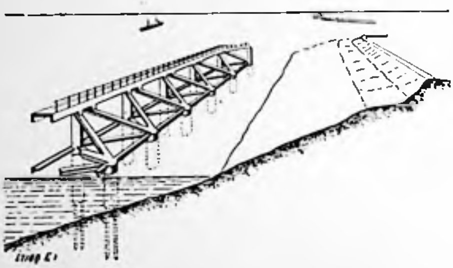


FIG. 79.—Deep-Water Quay No. 6, Thames Haven.

In the design of deep-water quays the conditions are so varied that anything like an analysis of cost is misleading. It appears, however, that for ocean-going shipping present-day costs in the Thames vary from £50 to £200 per foot run,

and recently in connection with preparing Port of London licence application plans for a proposed cold storage jetty involving some heavy dredging, the estimated cost exceeded the latter figure.

In all designs for tidal deep-water quays the following requirements must be kept in mind:—

1. Good facilities for mooring vessels.
2. Rigidity of structure.
3. Immunity from damage.

It is of convenience to deal with the second and third points first. As regards "rigidity" of structure, design has to be governed principally by experience, and we frequently have to deal with the problem of berthing ships in the Thames which, when laden, represent a gross weight of 35,000 tons. When alongside, wind pressure and tide pressure can be calculated, but these stresses are negligible compared with those sometimes caused in berthing a ship, and these latter stresses are, unfortunately, not calculable. Where reinforced concrete is adopted, wide spans coupled with the "cylinder" type of construction probably represent the best practice to-day. Beyond this generalisation, however, the question of rigidity is one which must be left to the individual engineer guided by his experience.

Immunity from damage is a big problem. Assuming that adequate fendering is provided, there remains the liability of serious damage from a large ship due to impact from berthing. In relatively protected tidal waters with tidal ranges, such as Antwerp or Rotterdam, this risk is reduced to a minimum if the ship is carefully handled.

In the Thames estuary, however, where relatively greater exposure and fast tides have to be dealt with, if the berth is of lesser length than the ship, it is essential that the ship, whether assisted by tugs or not, should present a definite angle to the face of the berth in coming alongside, since there is no "flexibility" of position open to her as in the case of long continuous quays. Many deep-water berths consist of a jetty head and dolphins, and the latter are therefore exposed to severe impact from either the port or starboard bow of the ship when berthing, and experience

shows that this class of berth is tending to become less suitable for the growing size of ships seeking berths in the open river outside the London Docks.

The first point enumerated above, however, principally merits attention, viz., "Good facilities for mooring." The author believes this to be the most difficult requirement to meet when dealing with large vessels in a tideway on a site exposed to heavy weather and with a long "fetch." This question must be considered from both the marine and civil engineering points of view. In one recent case designs were prepared for a continuous quay frontage some 1,350 lineal ft. in length. The range of tide was nearly 20 ft., and the mooring of large ships against such a continuous quay face was considered to present great practical difficulties. It was felt that the "spring" ropes could be dealt with, but that the bow and stern ropes would have to be increased in number and carried at awkward angles to compensate for the rise and fall of tide. There appears to be some doubt also if a ship so moored could be regarded as sufficiently held against the jetty. Unless this can be secured delays in loading or discharge will occur. In the case of a jetty and dolphins, as the ship's stem and stern generally project beyond these the bow and stern ropes can be carried ashore, a perfectly satisfactory method where it can be adopted. In all designs co-operation with officers of the mercantile marine as to lay-out is essential.

In the event of establishing entirely new industrial undertakings on the river-side the question of foundations is of importance. To some extent this is liable to be lost sight of by engineers who are perhaps too inclined to deal only with the problem from a purely technical point of view, and as one presenting technical difficulties only to be overcome. The practical engineer, however, should at all times bear in mind the financial aspect of any industrial undertaking, and wherever bad foundations can be avoided a corresponding large capital expenditure can generally be saved on which otherwise a large sum of interest has to be set aside yearly, resulting in a big and permanent handicap to economical production. In one such case a wrong choice of site resulted in excess expenditure of approximately £225,000, nearly

one-quarter of the whole cost of constructional work. In the case of factories on the river-side, where the ground is bad, there has been too much tendency to seek for small economies by placing the site of factory buildings as close to the river and wharf accommodation as possible. In many cases the subsequent working of the factory has shown this to have been an initial error, as the normal rate of interest on the excess cost of foundations often proves to be a greater charge than double handling and trucking or conveying the raw or finished materials by more modern methods. In another case the foundations of the factory building and warehouses were so bad that, although the principal vertical members had been carried on piling, the sills and flooring of the factory subsided throughout a large section of the factory buildings, and the intake and outfall works were affected.

On one site the author's firm was requested to advise on the question of increasing the wharf area. This wharf was some 600 ft. long, built into the river with two return ends, and it had been suggested by the owners of the factory that the enclosed area might be reclaimed. The wharf was constructed in 1905, and subsequently several attempts were made to fill in the area by the dumping of solid material. This was abandoned when indications were obtained that if dumping were continued the lateral stresses on the existing wharf might cause the latter to collapse outwards. The actual area of reclamation was in all about 102,000 sq. ft., and from a casual observation this reclamation appeared a simple problem. Borings were originally taken over this ground in 1905, and showed uneven strata and bad ground. One of the borings was fairly representative of the bad nature of the ground. This boring gave the following results:—

0 ft. to 15 ft. chalk (artificial tip).
At 20 ft. depth, clay.
" 25 ft. depth, peat.
" 30 ft. depth, old empty pipes.
" 35 ft. depth, soft drab clay.
" 40 ft. depth, very moist clay.
" 45 ft. depth, Thames ballast.

The Thames ballast was by no means at a consistent level, as landwards of this boring red sand and clay were met with, the ballast probably disappearing altogether with the clay and sand deposits resting on the underlying chalk. The situation was further complicated by the varied nature of the waste materials with which spasmodic attempts had been made to recover this ground.

Any attempt at a solid fill would have required an elaborate retaining wall with piled foundations carried right down to the ballast (or to the chalk), as the lateral earth stresses set up would have been high and increased by the normal wharf-load, which in this case was as high as 10 cwt. per square foot. The cost of doing this was estimated to be £122,160, and the scheme as it stood had therefore to be entirely abandoned. To pile the area all over and construct light staging would have cost at least £50,000.

It was then decided to test the area with a few trial piles, which duly confirmed these estimates, and as a result, the whole idea of reclamation was abandoned—a classic example of an error in the initial selection of a factory site.

One important point in connection with the carrying out of river contracts is the keeping of records in regard to the progress of work. The necessity for such records cannot be emphasised too much, and the cost of keeping them is negligible when compared with the great reductions in labour charges which follow.

One regrets that, owing to the special nature of marine work, delays are likely to occur in construction owing to unexpected causes being encountered, such as abnormal weather, loss of falseworks by collision, etc., and as time is usually the essence of the contract, any undue delay must be corrected at once and lost time made up for if possible. This can only be done by maintaining a complete series of progress diagrams which can seldom be kept in too great detail. In cases where the works are being carried out by a contractor, the engineer and the contractor should prepare together a definite programme of work and a time-table; and when the progress of the work falls behind this standard rate, a conference between the consulting engineers, the

principals, the contractors, the resident engineer, and the contractors' agent should always take place.

The author has found this policy sometimes looked upon as unnecessary by both clients and contractors in the early stages, but if the result as to time and cost is to give satisfaction to all concerned, no delay, even if apparently unimportant, should be allowed to pass without close and immediate examination as to cause.

It is well known there are two methods of keeping progress diagrams, namely:—

1. By preparing skeleton drawings showing all units of the work, which latter are marked off or coloured as they are finally fixed in position; or

2. By listing all the units of construction, and preparing curves or "graphs" for each unit.

The latter method is to be preferred for marine works of any extent, since the principal objection to the first method is the very elaborate series of drawings required; such drawings have to be of special character, since the contract drawings themselves, though indicating the exact nature of the work, will not necessarily repeat any uniform section of the work more often than is necessary for constructional purposes, whereas progress diagrams must show every unit as often as it occurs, whether it is concrete block, pile, waling, bracing, diagonal, deck bearer, girder, etc. The second method has also the advantage of enabling whoever is responsible for maintaining the diagrams up to date to plot the principal curves on one sheet, although varying scales invariably have to be applied to each curve. A standard tracing, showing the correct rate of progress for each unit, can be superimposed from time to time over the actual records, and if the working curves fall away or unduly exceed the rate of progress on the standard tracing, it is obvious that steps must immediately be taken to correct the rate of progress of this particular section of work.

The above precautions are frequently looked upon as too academical to be applied in practice. The author wishes to oppose this view, as he believes, from the result of actual

experience in contracts involving large sums, that progress diagrams have been of direct and practical value in indicating at once that a section of the work had dropped behind the remainder, a fact not at all obvious from mere inspection.

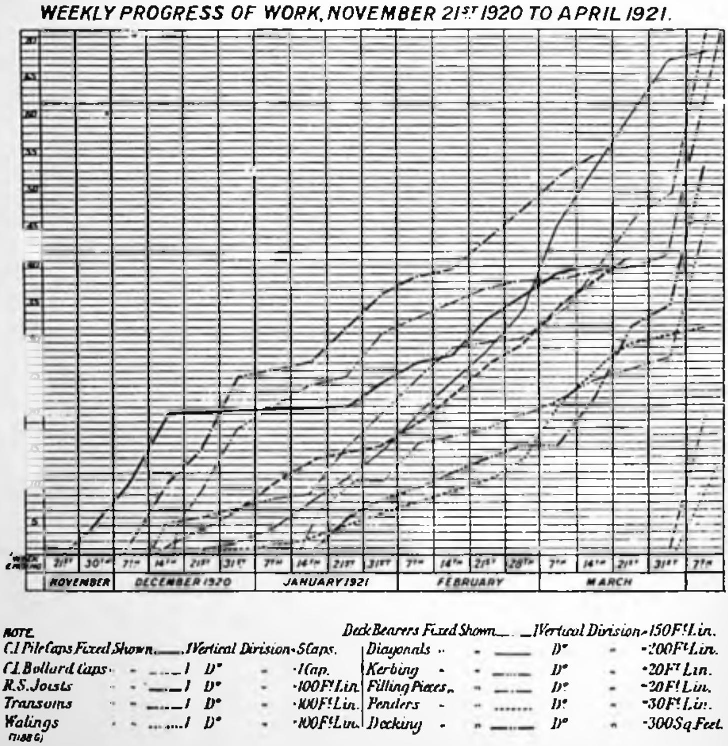


FIG. 80.—Example of Progress Diagram for Marine Works.

The consultant in making periodical inspections of work can thus go on the site with some idea of progress in his mind, and is often able to discover the causes of delay with greater facility. The important point to remember is that the progress diagram indicates beforehand where delay may be expected in the future, and not merely the extent to which the works are behind at the moment of inspection.

It is hoped that the above résumé on progress diagrams will serve the purpose intended, and the notes on this subject are therefore concluded by a reproduction (Fig. 80) of actual records kept by a resident engineer (Mr L. G. Carter) in 1920 to 1921, in connection with an extensive deep-water pier constructed in timber and steel on the River Thames.

FIG. 81.—Breaches at Fingrinhoe Marshes, 1922.

APPENDIX

LEGAL ASPECTS OF MARITIME ENGINEERING

It is, of course, not within the province of the consulting engineer to advise on questions of law. He requires, however, a working knowledge of contractual law as applied to public works for the very simple reason that, without such knowledge, he is liable to overlook the results which may accrue from the construction of marine works not in themselves directly detrimental to his client's interests but which may render them liable to legal processes by other parties.

The ownership of any foreshore and the proper definition of the word "foreshore" are important. The word "foreshore" covers the area lying between the contours of ordinary high and low tides. These contours are the mean levels respectively of ordinary Neap and ordinary Spring Tides. The determination of these two contours is a delicate process based on observed tidal levels throughout the year of the tides occurring on the fourth tide after the "full" and "change" periods of the moon (Spring Tides), and of the intervening Neap Tides.

One of the standard works of reference of use is the latest edition of "The History of Law of the Foreshore and Seashore"—Stuart A. Moore and R. G. Hall—published by Messrs Stevens & Haynes. This book, while doubtless a standard work from the legal point of view, is somewhat obscure on the subject of tides, and there seems some confusion of thought between the terms "Neap" and "Spring Tides." These tides occur in fortnightly cycles with, however, increased effect in spring and autumn. There are really six littoral contours recognised by the law, viz. :—

1. High-water Ordinary Spring Tides.
2. High-water Ordinary Tides.
3. High-water Ordinary Neap Tides.
4. Low-water Ordinary Spring Tides.
5. Low-water Ordinary Tides.
6. Low-water Ordinary Neap Tides.

It is quite clear that, owing to variations caused by wind, estuarial discharge, and other causes, no *exact* contour lines can be drawn, and the limits of foreshore ownership are therefore always vague. The

definition above does enable the engineer in some cases to determine definitely whether a given point *does* or *does not* lie within the foreshore limits. On the other hand, the physical vagaries of the tide in some cases may compel the engineer to acknowledge that a given point *may* or *may not* be within the foreshore limit.

There lies, therefore, round our coasts an indeterminate strip of land known as the "foreshore." The ownership of this foreshore was originally vested in the Crown, but the latter has in many cases ceded the rights in the form of grants, so that the ownership may be vested in the lord of the manor or in the local authorities. Irrespective of the title of ownership, the Marine Department of the Board of Trade exercise statutory control of these lands, which precludes any constructional work being undertaken without their sanction, or the dumping or removal of beach material. The control of the Board of Trade is exercised principally in the interests of navigation. Proposed works submitted for approval to the Board of Trade are referred by the latter department to the Ministry of Health, who have jurisdiction in the case of public expenditure or presumably in cases where the public interest is at stake.

The foregoing position may be varied in rivers and estuaries under a conservancy such as the Port of London Authority, in whom all authority is centred by Act of Parliament.

The sudden accretion of land, by legal precedent, places such land under the ownership of the Crown, while gradual accretion increases the holding of the landowner concerned.

The foregoing summary may be relied on as substantially correct, but the subject is an obscure one, and, by virtue of the fact that scarcely any two sites are identical as to conditions, there is not a great deal of precedent to rely on.

Civil engineers are frequently retained by wharfingers, shipowners, or insurance companies to resist claims for damages or to formulate such claims and assess costs of reparation. As such surveys should always be made without prejudice to a client's interests, it is of great importance, during a joint survey, to agree to nothing beyond the *facts* of a case, the question of liability not being one for the engineer to discuss, and any such expression of opinion on his part may be held to be evidence, and may be erroneous or damaging to his client.

It is to be feared that no engineering text-book can provide legal data of great value to the engineer, but the following table of cases, with most of which the author has been directly or indirectly concerned, may offer parallels to other cases. In this list are included investigations of interest to the maritime engineer; some of them have been dealt with by the High Courts, others settled out of Court, and some have never reached the stage of litigation, but were disputes, nevertheless, of importance.

Nature of Case.	Date.	Summary.	References.
Alleged foul berth, East Greenwich.	1921	The S.S. *Nicolaos Zafirakis*, with nearly 6,000 tons of coal, took the ground at a steel jetty at East Greenwich and became strained at low water. The berth had not been periodically examined, and this omission cost the jetty owners heavy expense in ship repairs, and nearly £5,000 in dredging costs.	Engineers' report to South Metropolitan Gas Co.
Tidal Hydro-Electric Power Scheme, Mersea Island.	1921	This proposal was investigated as to its practicability. It was found that one arm of the sea filled during high tide could discharge into another arm of the sea from which the tide was to be excluded by a barrage. This scheme had some attractions, but the relatively small power available, and the existence of valuable oyster fisheries which would be exterminated, were factors against the enterprise. There was some doubt as to whether Parliamentary powers would be required.	Deacon & Co., Solicitors.
Chapel Cleeve foreshore, sea defence works.	1921	An interesting case of a short sea-wall under 100 ft. in length, constructed by the landowner at a cost of nearly £3,000, to protect a couple of small rural cottages.	Gerald Lysaght, Esq., Chapel Cleeve, Somerset.
Somerset County Council sea defence works.	1921	Extensive protective work on the foreshore between Minehead and Watchet was sanctioned by the Ministry of Transport and carried out. The case is of interest, as indicating the liability of a County Council to maintain a little used coastal county road, thereby involving very heavy expenditure.	Reports of public inquiry, Ministries of Health and Transport, Minehead, 30th April 1921.

Nature of Case.	Date.	Summary.	References.
Extension of boundaries of Sewer Commissioners.	1921	The Fobbing Level Commissioners were requested by the Board of Agriculture, under the terms of the Land Drainage Act, 1918, to extend their boundaries inland and deal with certain land drainage questions in addition to their functions of maintaining the river walls and marsh ditches.	Clerk to the Fobbing Level Commissioners.
Repeated damages to reinforced concrete jetty, Dagenham, Essex.	1921	This jetty was utilised by the Port of London hoppers for bunkering purposes. Several claims were made by the jetty owners from time to time, for minor damages to the jetty in respect of piles and bracing broken when the hoppers berthed. This constantly recurring liability ended in a recommendation to the underwriters to require efficient fendering to be provided.	The Salvage Association.
Damage to timber jetty, W. Thurrock, Essex.	1920	A peculiar accident, primarily arising from indefinite berthing instructions. There are two jetties on this site, and the reinforced concrete lighter *Cretecamp* was ordered to take the *outer* jetty, and her master selected the downstream jetty—not the one intended. She had to change her berth, and in so doing damaged the piling of the upstream jetty.	The Salvage Association.
Failure of sea defences, Leysdown, Isle of Sheppey.	1920	A curious case, in which the landowner sought to recover the cost of extraordinary damages to his sea-wall, caused by shock and vibration from continuous practice of bombing with high explosive shells on the adjacent foreshore. While the vertical subsidence of the wall foundations supported this view, the principal difficulty was the unknown behaviour of a dense clay formation under these circumstances. The recent researches of Mr A. S. E. Ackermann, B.Sc., and Dr Langtry Bell have thrown some light, however, on this.	J. D. F. Andrews, Esq., Isle of Sheppey.

Nature of Case.	Date.	Summary.	References.
St Osyth sea defences, Essex.	1920	Blockhouse: Wick Farm Marshes lie below high-water mark, and the protective earthen embankments had been badly eroded and the old timber facings were failing by decay. Recommendations were made for a reinforced concrete facing, and the situation was of interest, as a breach in this wall would possibly be followed by a diversion of Brightlingsea Creek, with disastrous effects to the town.	O. B. Smith, Esq., Braintree, Essex.
Erosion of foreshore, Burnham, Somerset.	1920	The flat sandy foreshore and sand dunes in this vicinity are unstable, and three landowners, uncertain as to the best course to pursue, combined in securing a technical opinion. This is one of the exceptional cases of adjacent owners combining in a common interest with the object of participating in a general scheme of defence.	W. F. Jacques. Solicitor, Burnham, Somerset.
Assessment of harbour accessories, Trèport, France.	1920	This was a case in which a shipping company desired to purchase a harbour undertaking, and required a valuation of warehouses, cranes, electric capstans, power plant, workshops, offices, etc., at short notice. There was no time for a complete inventory, and an approximate valuation of 2,832,500 francs was made at a week's notice. The case is interesting (1) as indicating the degree of responsibility which a civil engineer is frequently called on to assume; and (2) as presenting difficulties as to arriving at an equitable fee.	...

Nature of Case.	Date.	Summary.	References.
Proposed bridge over River Neath, Briton Ferry, Glamorganshire.	1919	Very considerable opposition by the Neath Urban and Rural Councils, railway companies, and landowners and local industries was offered to the proposal to construct a swing bridge at Briton Ferry. The Bill was promoted by a powerful corporation owning sites on both sides of the river. One of the most interesting side issues to this case was the possibility or otherwise of diverting the river channel by means of tipping steel slag.	Edward Powell, Solicitor, Neath, South Wales.
Collision between steamer and dock wall, Tilbury.	1918	This case involved a settlement with the Port of London Authority in respect of damage by stem-on collision to the brick and granite dock wall at the entrance to Tilbury Docks.	Messrs Thomas Cooper & Co., Solicitors to the owners of S.S. *Maimyo.*
The sea defences of the Dublin & South-Eastern Railway.	1917 1918	During the Great War this line was of peculiar commercial and strategical importance, and the Government of Ireland and the railway company sought expert opinions on the question of the best means of maintaining traffic over 11 miles of line exposed to the ravages of the sea. There was some conflict of opinion.	Reports of (1) Sir Maurice Fitzmaurice, P.P.Inst.C.E.; and (2) A. E. Carey, M.Inst.C.E.
Collision between steamer and reinforced concrete jetty, Purfleet.	1917	The S.S. *Navigator* (Messrs T. & S. Harrison) collided with a reinforced concrete jetty owned by the Steamship Owners' Coal Association. The principal interest in this case was in regard to the high cost of repairing this class of structure. The claim for constructional work reached £9,250, apart from consequential damages.	Messrs Pritchard & Sons, Solicitors to the owners of S.S. *Navigator.*

Nature of Case.	Date.	Summary.	References.
Proposed sea defences, Thames Haven, Essex.	1917	In this case three landowners, viz., the Anglo-Saxon Petroleum Co., the Midland Railway Co., and the London & Thames Haven Oil Wharves Ltd., sought advice on sea defence works as the result of representations made by the Fobbing Level Commissioners, as to the necessity of expenditure on these three contiguous frontages. The case is an interesting example of frontagers combining in a common cause—a somewhat rare occurrence.	Report of Messrs Coode, Matthews, Fitzmaurice, & Wilson to the Midland Railway Co.
Proposed coast defence works, Rhyl to Prestatyn, North Wales.	1914	In this case the Flintshire County Council and certain landowners combined to obtain advice on coast defence works. Recommendations were made including a sea-wall and groyning and the construction of a marine drive along the coast sand dunes. This was another case of useful co-operation.	Report by A. E. Carey, M.Inst.C.E., to the Flintshire County Council.
Clacton Pier, underwriters' risks.	1914	The inspection of this structure as to condition is an interesting case of the civil engineer's responsibility in advising as to insurance risks. The pier was subsequently reported on as to condition after occupation by the Admiralty as a temporary mine-sweeping base.	...
Arbitration on compulsory acquisition of coastal land.	1914	The War Office acquired the peninsula of Orfordness as a flying ground. Extra evidence was called by the War Office as to the condition of the sea and river defences. The case was of interest to landowners in respect of the potential depreciation of low-lying land subject to inroads by the sea.	Messrs Cherer & Co., 8 New Court, Carey Street, W.C.

Nature of Case.	Date.	Summary.	References.
Proposed reparation of breach in river embankment, River Deben, Suffolk.	1914	Three proposals by an engineer and two contractors clearly demonstrated that the cost of repairing the breach in this clay embankment was out of all proportion to the value of the land which would thus be reclaimed. The project was therefore abandoned.	J. Lomax, Esq., Woodbridge, Suffolk.
Intervention by Level Commissioners in respect of coast erosion, Frinton.	1913	The Tendring Level Commissioners maintained that piling driven into the foreshore by the Frinton U.D.C. caused erosion at the toe of the Commissioners' wall. The case was heard in Chambers, and a settlement arrived at, the local Council agreeing to reinstate the wall.	"Law Reports," Morton *v.* Frinton-on-Sea U.D.C.
Flooded marsh lands adjacent to a river.	1913	Continuous pumping by the Great Western Railway, in connection with the Severn Tunnel, caused subsidence of 200 acres of marsh land, 3,500 ft. away from the River Severn. The case was of interest in respect of which was the most efficient method of draining these lands.	"Maintenance of Foreshores" (Crosby Lockwood, 1914).
Subsidence of warehouse on wharf, Lot's Road, Chelsea.	1913	Certain work in connection with condenser circulating water intake and outfall necessitated deep excavation for the screening chamber. An adjacent wharfinger claimed damages for subsidence of a warehouse through excessive underdrainage caused by pumping. The claim was resisted.	Messrs Cochrane & Sons, contractors.
Security of mortgages on property subject to erosion.	1913	The legal aspect of this case was unusual and important. The property involved was a hotel situated on cliffs, being eroded at the toe by the sea, and therefore threatened with destruction.	...

Nature of Case.	Date.	Summary.	References.
Sand travel at Burry Port Harbour, Carmarthen, South Wales.	1913	The harbour was at this period threatened with obstructions by sand banks at its entrance. The case was principally of interest in the study of sand movements in the Burry inlet, and the liability of the owners of an adjacent breakwater to maintain same.	The Burry Port and Gwendraeth Valley Railway Co.
Displacement of bridge pier, Wandsworth Bridge, S.S. *Wandle.*	1912 1913	The S.S. *Wandle*, owned by the Wandsworth, Wimbledon, & Epsom District Gas Co., was one of the largest vessels navigated in the upper reaches of the River, Thames; she collided in fog with one of the steel columns of Wandsworth Bridge and displaced it. The column was fractured 18 ft. above its foundations in London clay, and ceased to support a wrought-iron continuous lattice girder of the bridge. The ship's underwriters and the London County Council had considerable difficulty in agreeing on the assessment of damages.	London County Council.
Assessment of damage to a timber pier, Purfleet, Essex.	1911	A timber jetty, constructed in 1904 at Purfleet by the British Petroleum Co., was run into at night by the S.S. *Tongariro* (11,000 tons), and the approach to the jetty head was cut in two, the ship suffering no damage. The case involved an attempt to assess consequential damages incidental to dredging another berth at Becton to accommodate shipping.	Messrs Thomas Cooper & Co., 21 Leadenhall Street, E.C., Solicitors to the British Petroleum Co.

Nature of Case.	Date.	Summary.	References.
Southend sewage outfall and pumping chamber works.	1910	The point at issue was the liability of the contractor, or otherwise, to make good considerable damage to works in course of construction as the result of the failure of timbering in deep trenches, and later as to the responsibility of the contractor to make water-tight joints in a sewer outfall by means of methods prescribed by the engineer, which methods the contractor maintained were impracticable. The question raised at all events showed the undesirability of a civil engineer specifying methods of construction.	Reports of A. E. Carey, M.Inst.C.E.
Coffer-dam collapse, Deptford Creek Bridge reconstruction.	1910	An interesting case of failure, originally attributed to defective steel piling. It was subsequently ascertained to be due to faulty timbering.	London County Council and British Steel Piling Co.
Alleged foul berth, River Thames.	1909	Owners of S.S. *Irkutsk* claimed extensive damage to the ship on taking bottom at Hayes' Wharf. A survey, however, showed a fair berth, and the claim was therefore not pressed.	The United Shipping Co.
Salvage case, damage by S.S. *Patris*, River Tyne.	1909	This was one of the most remarkable salvage cases of recent years. The S.S. *Patris* (2,500 tons) was launched from the yard of the Northumberland Shipbuilding Co. A drag-box shackle broke and she crossed the river stern first, entering a recently flooded drydock of the Mercantile Dock Co., carrying away gates and damaging the ship within. The assessment of damages, which were extensive, was an intricate and difficult matter.	Messrs Thomas Cooper & Co., Solicitors to the under-writers.
The "blowing" of a coffer-dam (Sunlight Wharf, London).	1909	The point at issue was the liability of contractors to make good damage caused by a "blow" into a coffer-dam from the river owing to excavation being carried too deep.	Report of A. E. Carey, M.Inst.C.E.

Nature of Case.	Date.	Summary.	References.
Scour caused by Marine Works, Purfleet.	1909	Under a private Act of Parliament defendants (The Tunnel Portland Cement Co.) were liable to contribute with others towards the cost of maintaining the plaintiffs' river wall. The plaintiffs (The Steamship Owners' Coal Association) constructed a pier which defendants maintained increased scour and consequent expenditure on wall. Judgment was given in favour of the plaintiffs on the grounds that they had obtained statutory powers to construct a pier, and were therefore not responsible for any detriment to other interests.	"Law Records," Official Referee. Messrs Hunt & Hunt, Solicitors to defendants.
Alleged failure of suction dredger to perform its guaranteed duties (Suffolk).	1908	This test case was of interest as showing the heavy liability involved under guarantees. The plant manufacturers supplied a suction dredger—separator, pumps, and floating pipe line. The plant was admittedly experimental, but guaranteed. It was employed on the River Blyth, Suffolk, and failed to discharge sludge onto the marsh lands sufficiently rapidly to enable the latter to be utilised for commercial purposes. This loss to the contractor was held to be part of his damages, and his counter-claim in respect of this was upheld by the Court.	"Law Reports," Gwynne & Fasey.
Contractor's claim for extras in repairing sewage outfall, Bridlington, Yorkshire.	1908	This test case of Bell *v.* Bridlington was of remarkable length and interest. It involved tidal work on a foreshore on a site not coinciding with contract drawings. The contractor claimed day work extras owing to his having to carry out the works in deeper water than anticipated. The plaintiff failed in that there was (1) no new contract, and (2) no additional contract under seal.	"Law Reports," Messrs Grundy, Lamb & Grundy, Solicitors, Manchester.

Nature of Case.	Date.	Summary.	References.
Failure of steering gear and consequent collision with a timber jetty, Purfleet.	1908	The S.S. *Harrovian* was proceeding down stream and ran ashore stern-on. Swinging round with the ebb tide she struck the Anglo-American Oil Co.'s timber jetty, severely straining same. Liability was admitted, and a settlement arrived at.	Anglo-American Oil Co. Ltd.
Damage to Tidal Dock, Barnard & Gabriel's Wharf.	1907	The ship became "mud sucked," and on refloating broke her moorings and damaged a timber dock face. The case contained some interesting data as to (1) liability of ship to have proper mooring ropes out, and (2) liability of dock owners to keep the dock clear of surplus mud.	"Law Reports," Messrs Pritchard & Sons, Solicitors to owners of S.S. *Narva*.
Damage to dock wall caused by steamer dragging moorings.	1907	A case of peculiar interest in regard to the liability of dock owners, Cardiff, to provide safe moorings. The owners of the S.S. *Llansannor*, defendants, succeeded in maintaining their defence on the grounds that a screw mooring should be capable of withstanding a definite stress. The intensity of this stress and methods of calculations were of great importance to the defence.	"Law Reports" (Admiralty Division), Messrs Donald MacLean & Hancock, Solicitors to the owners of S.S. *Llansannor*.
Rights-of-way along foreshore (Pierson *v.* Burnham, Somerset, U.D.C.).	1907	This is regarded as a test case. The case was decided in favour of the plaintiff, who advanced his fences as the sea receded. Subsequent erosion prevented access along the foreshore, and the public threatened to destroy the fences.	"Law Reports" and pamphlet by D. W. F. Jacques, Solicitor, Burnham, entitled "The Foreshore."

Nature of Case.	Date.	Summary.	References.
Salvage operations, Admiralty Harbour, Dover.	1906	The south breakwater was under construction at this time. Two heavy gantries on piling carried Goliath and Derrick cranes over transverse spans of 105 ft. These falseworks were most elaborate, and consisted of timber piling and steel lattice girders. A small Danish timber ship, of about 1,100 tons, ran into the outer gantry in a fog, demolishing three bays of the seawards gantry, owing to the heavy blue gum piles fracturing at the scarfing joints. The wreckage was ultimately cleared with blasting gelatine fixed by divers and detonated electrically from above. The damage exceeded £30,000, and the case is interesting because, although the ship was responsible, her statutory liability was limited to £8 per ton, and the bulk of the loss, therefore, fell on the underwriters.	The Salvage Association (Surveyors' Report).
Acquisition of ferry rights, Littlehampton.	1906	The Corporation of Littlehampton decided to purchase the ferry rights of Mr G. M. Percy in respect of the ferry across the River Arun. The case went to arbitration, and the high figure of £8,415 was the amount assessed by the arbitrator as the value of the ferry rights.	Daniel Watney, Esq., Arbitrator, Surveyors' Institution, May 1906.
Salvage case, Thames Haven, Essex.	1906	The S.S. *Lackawanna* destroyed a steel pier head at Thames Haven, the property of the London & Thames Haven Oil Wharves. The Company's claim was settled for £6,500. The case is worth recording in view of the decision, ultimately carried out, to reconstruct this jetty in reinforced concrete.	Report of A. E. Carey, M.Inst.C.E.

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